PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 12, NO. 5 ▽ JULY, 1931

A FOUR-LANE HIGHWAY IN PENNSYLVANIA
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THE SUBGRADE SOIL CONSTANTS, THEIR SIGNIFICANCE, AND THEIR APPLICATION IN PRACTICE

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, A. M. WINTERMYER, Assistant Highway Engineer, and E. A. WILLIS, Assistant Highway Engineer, U. S. Bureau of Public Roads

PART II: A DISCUSSION OF THE SOIL CONSTANTS AND THE SOIL IDENTIFICATION CHART

THE FIRST part of this series of articles, published in the June, 1931, issue of PUBLIC ROADS, dealt with the five major physical properties of soils, internal friction, cohesion, compressibility, elasticity, and capillarity, and their relation to subgrade performance. It was shown that the presence of these properties in varying degrees depends very largely on the soil constituents, although the state in which a soil exists is a factor which can not be neglected. A method was outlined for classifying soils according to the predominance of certain constituents in them, and therefore, in a general way, according to their physical characteristics. The present report (Part II) discusses the test constants which serve to identify the constituents of soils and their resulting properties; and the soil identification chart, which is a convenient means of analyzing the data given by the laboratory tests. Part III describes the manner in which the soil identification chart is used in actual practice.

MECHANICAL ANALYSIS LIMITED AS AN INDICATOR OF SUBGRADE EFFICIENCY

From the information given in Part I it is clear that a knowledge of the size of soil particles does not furnish a complete identification of subgrade characteristics. Indeed, the mechanical analysis is very much limited in its ability to disclose accurately the size of the smaller soil particles, which must be determined by means of some method of sedimentation instead of by sieves. As a consequence, for grains smaller than about 0.074 millimeter (No. 200 sieve) in diameter, the grain diameters given by the mechanical analysis instead of being the diameters of the grains tested are the diameters of spheres which, according to Stokes law, (*20*), settle in water at a rate equal to that of the particles of soil being analyzed. A further inaccuracy is introduced by the fact that the rate at which small soil particles settle depends upon the extent to which the soil is dispersed; and this in turn depends upon both the method of agitation and the chemical reagent used in dispersing the soil.

Thus, for instance, a soil which, according to the mechanical analysis, contains 50 per cent of clay, does not necessarily contain 50 per cent of particles having diameters smaller than 0.005 millimeters. The correct interpretation of the test is that 50 per cent of the particles settle through water at a rate equal to that of spherical particles not exceeding 0.005 millimeters in diameter.

Figure 34,[7] which shows photomicrographs of soil suspensions at different periods of time after dispersion, discloses how very much the shape of soil particles is likely to vary from the spherical.

Table 4, furnished by L. B. Olmstead, L. F. Alexander, and H. E. Middleton (*20*), illustrates how the type of dispersing agent influences the silt and the clay contents furnished by the mechanical analysis. As shown in this table, the value obtained for Houston black clay may be 45 per cent or 64 per cent, depending on whether sodium hydroxide or sodium oxalate is used as a dispersing agent. Furthermore, the clay fraction depends upon the degree of agitation used during dispersion. Too little fails to separate the particles sufficiently, causing the indicated clay content to be too small. Too much agitation not only separates the particles but may also break them into smaller pieces, thus causing the indicated clay content to be too large.

TABLE 4.—*Yield of clay obtained by use of various dispersing agents* [1]

Soil type and source	Dispersing agent							
	Ammonia		Sodium hydroxide		Sodium carbonate		Sodium oxalate	
	Percentage of clay particles obtained							
	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.	0.005 mm.	0.002 mm.
Wabash silt loam (Nebraska)	30.7	24.1	33.2	28.5	33.1	28.3	34.5	29.6
Houston black clay (Texas)	38.4	26.4	44.5	30.6	61.3	53.5	63.8	53.8
Carrington loam (Iowa)	7.8	6.4	24.9	21.8	23.4	19.9	24.7	22.9

[1] Average of duplicate determinations.

It should be remembered also that when grain size is computed from a knowledge of rate of settlement of particles in water, it is assumed that all of the grains in the soil mass being tested possess equal specific gravities, and this is not necessarily true.

The fact that mechanical analysis is inadequate to identify the characteristics of subgrade soils is illustrated by the gradings typical of the uniform subgrade groups, which are given in the following paragraphs. The two terms, "effective size" and "uniformity coefficient," used in this discussion, are defined as follows:

The effective size is the maximum size of the smallest 10 per cent, by weight, of the soil particles. On a soil accumulation curve (fig. 35) the value of the effective size is given by the abscissa of the point on the curve having the ordinate 10 per cent.

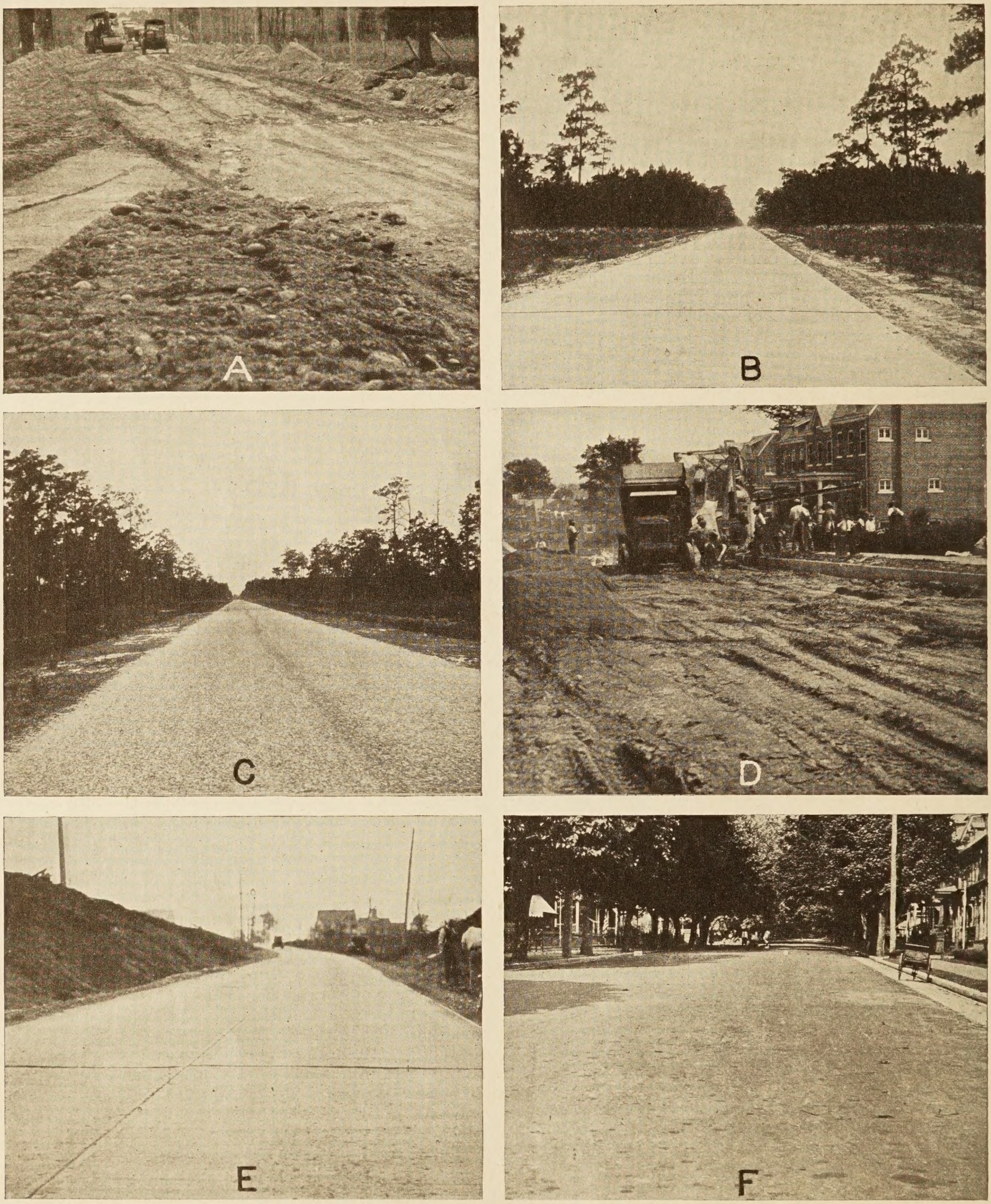
The uniformity coefficient is the ratio between the maximum size of the smallest 60 per cent, by weight, of the soil particles, and the effective size. Its value may be obtained from a soil accumulation curve by computing the ratio between the abscissa of a point whose ordinate is 60 per cent and the abscissa of a point whose ordinate is 10 per cent.

The typical gradings are as follows:

Group A–1 subgrade.—Material retained on the No. 10 sieve, not more than 50 per cent. The soil mortar, that fraction passing the No. 10 sieve, composed as follows: Clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand (retained on the No. 60 sieve) 45 to 60 per cent. Effective size approximately 0.01 millimeter and uniformity coefficient greater than 15.

The soil accumulation curve of Figure 35 shows graphically the average grading of stable soil mortars. In this case the

[7] The numbers of figures, tables, footnotes, and equations are consecutive with those of Part I of this report (Public Roads, June, 1931). Italic numerals in parentheses refer to the bibliography given at the end of Part I.



Examples of Subgrade Classification and Pavements Laid on Different Types of Subgrade. A, Base Course Being Constructed of Group A–3 Material; B, Concrete Pavement Laid on Group A–3 Subgrade in Florida; C, Bituminous Macadam Wearing Surface on Group A–4 Base. D, Group A–5 Subgrade Showing Rebound After Being Rolled. E, Excellent Concrete Pavement Constructed on Group A–5 Subgrade. F, Surface Treated Macadam Constructed on an Excellent A–6 Subgrade.

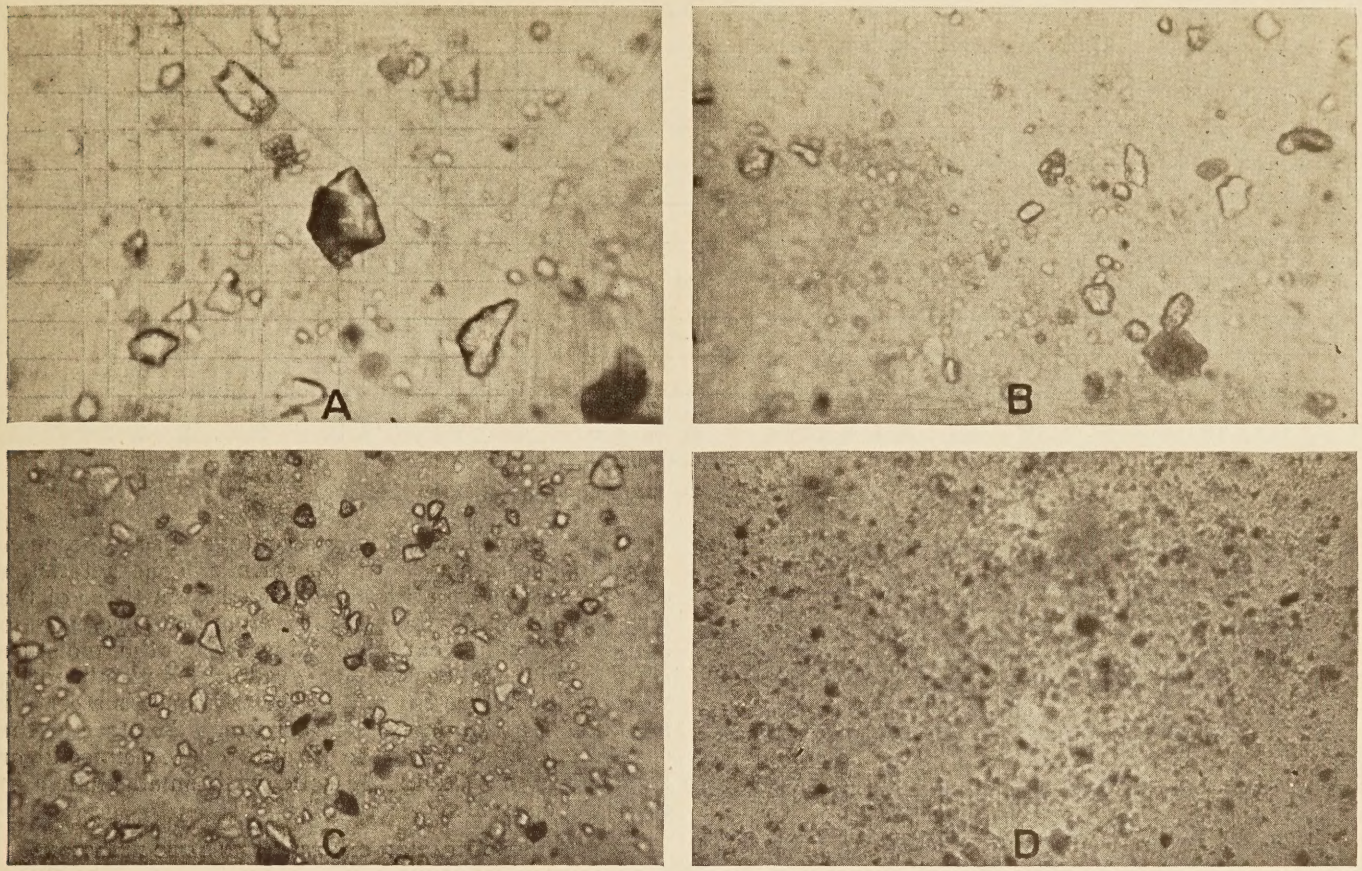


FIGURE 34.—PHOTOMICROGRAPHS OF SOIL SUSPENSION AT DIFFERENT PERIODS OF TIME AFTER DISPERSION: A, AFTER 1 MINUTE; B, AFTER 2 MINUTES; C, AFTER 5 MINUTES; D, AFTER 15 MINUTES. NOTE REDUCTION IN SIZE OF PARTICLES IN SUSPENSION AS TIME OF SEDIMENTATION INCREASES

effective size is 0.01 millimeter and the uniformity coefficient is $\frac{0.43}{0.01}$, or 43.

Group A-2 subgrade.—Not less than about 55 per cent of sand in the soil mortar.

Group A-3 subgrade.—Effective size not likely to be less than 0.10 millimeter.

Group A-4 subgrade.—Likely to contain sand in amount less than 55 per cent.

Group A-5 subgrade.—Same as group A-4.

Group A-6 subgrade.—Likely to contain more than 30 per cent clay.

Group A-7 subgrade.—Same as group A-6.

Group A-8 subgrade.—Grading not significant.

From the facts given in the preceding discussion it is plain that mechanical analysis gives only approximately the diameter of soil particles of small size, and that the size of grain, even if accurately known, is a very imperfect criterion of subgrade soil characteristics. In order to identify these characteristics it is necessary to employ constants which disclose the degree to which particular physical properties are present in a given soil.

SIGNIFICANCE OF TEST CONSTANTS DISCUSSED

Among the constants which have been suggested as aids in identifying the important subgrade properties are the liquid limit, the plastic limit, the plasticity index, the shrinkage limit, the centrifuge moisture equivalent, the field moisture equivalent, the shrinkage ratio, the volumetric change, and the lineal shrinkage. In Table 5 are listed values of these test constants for a group of representative subgrade soil constituents. In the tests from which the values given in Table 5 were derived the following materials were used:

Sand.—Potomac River sand; that fraction passing the No. 20 and retained on the No. 100 sieve.

Silt.—Silty sand soil obtained in Rock Creek Park, District of Columbia.

Clay.—Yaguajay clay (clay about 70 per cent and silt about 30 per cent) from Cuba. Furnished by H. H. Bennett, United States Bureau of Chemistry and Soils.

Colloids.—Bentonite. According to C. S. Ross and C. V. Shannon (*19*), out of five clay minerals contained in bentonite, three are micaceous, one is platy crystaline, and one is amorphous. According to the hydrometer analysis 99 per cent of the bentonite particles are smaller in diameter than 0.05 millimeter (silt), 85 per cent are smaller than 0.005 millimeter (clay), 80 per cent are smaller than 0.001 millimeter (colloids), and 79 per cent are smaller than 0.0005 millimeter.

Mica flakes.—That fraction passing the No. 20 and retained on the No. 100 sieve.

Diatoms.—Celite: 95 per cent of particles smaller in diameter than 0.05 millimeter and 61 per cent smaller than 0.005 millimeter.

Peat.—Everglade peat, Florida, furnished by the United States Bureau of Chemistry and Soils. Sixty-five per cent of particles smaller in diameter than 0.05 millimeter and 18 per cent of particles smaller than 0.005 millimeter.

In addition to the tests referred to above both a compression and a slaking test may assist in identifying those binder clays of the group A-1 and A-2 subgrades which, because of certain chemical constituents, are inclined to "set up" upon drying. These same tests performed upon soil samples both treated and untreated with bituminous materials serve to disclose to some degree the increase in stability furnished by the treatment. Chemical tests may also be required to disclose those soil chemicals which are likely

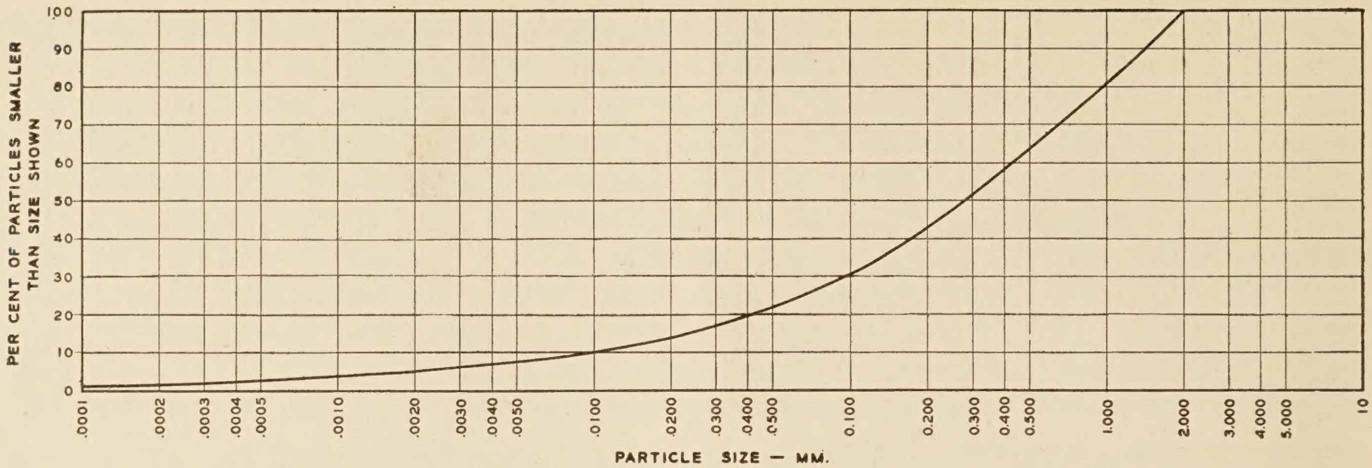


FIGURE 35.—GRAIN SIZE ACCUMULATION CURVE FOR GOOD BINDER

TABLE 5.—*Laboratory test results given by representative soil constituents*

Soil constituent	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Volumetric change	Lineal shrinkage
			Limit	Ratio	Centrifuge	Field		
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*	*Per cent*	*Per cent*
Sand	20	[1] 0	------	------	4	25	0	0
Silt	27	7	19	1.8	22	22	5	2
Clay	100	54	11	2.1	70	55	92	19
Colloids	399	354	6	2.0	(2)	86	160	27
Colloids, 50 per cent	174	154	12	1.9	291	54	------	------
Mica	123	0	160	0.52	159	142	[3] −9	[3] −3
Diatoms	163	0	134	0.5	221	212	39	10
Peat	136	0	44	0.9	90	121	69	16

[1] Indicates nonplastic soils without plastic limits.
[2] Centrifuge moisture equivalent could not be determined because of very great expansion suffered by pure bentonite when being saturated.
[3] Negative value indicates expansion of mica on drying, discussed subsequently.

to exert a detrimental influence upon concrete pavements and structures. The compression, slaking, and chemical tests are not discussed in this report.

In order to use the constants intelligently one must thoroughly understand their physical significance. The soil identification chart, discussed subsequently in this report, assists in readily identifying the characteristics of many soils. The interpretation of results furnished by tests performed on many other soils, however, can be accomplished only by an intimate knowledge of physical phenomena occurring when the soils are subjected to test. For this reason an attempt is made to explain these physical phenomena in detail.

In this connection, it should be noted that the significance of a constant may vary, depending upon the degree of capillarity possessed by the soil. For this reason soils are referred to as either expansive or nonexpansive soils, according to their degree of capillarity.

The expansive soils, silt, clay, etc., are those whose capillarity is sufficient to cause swelling, shrinkage, or detrimental frost heave in appreciable amount. The nonexpansive soils are those generally termed sands, whose capillarity is not sufficient to produce these phenomena.

It is important to remember also that when a nonexpansive soil is being added to an expansive soil in increasing amounts the change in certain properties from the expansive to the nonexpansive variety may be abrupt instead of gradual. As will be shown later, sand added to clay in increasing amounts causes the resulting mixture to change its shrinkage characteristics very slowly until, when the percentage of sand becomes equal to a certain amount, the shrinkage characteristics of the mixture are abruptly changed from those of the clay to those of the sand. Important decrease in supporting value caused by increasing the moisture content of soils occurs not gradually but abruptly, when the moisture content of the soil exceeds a certain amount. In the same abrupt manner the state of the soil may change from the plastic to the semisolid or from the semisolid to the solid with small change in moisture content.

SOIL IDENTIFICATION CHART SHOWS BASIC RELATIONS

The chief value of the soil constants as a means of identifying subgrade soils lies in the relations existing between them rather than in the magnitudes of the individual constants, considered separately. The use of these interrelationships, combined with the values of the constants themselves, as a basis for the construction of the soil identification chart is a distinctive feature of the procedure developed by this bureau in its subgrade studies.

The four graphs of Figure 36 constitute the soil identification chart. They show the relations which have been found experimentally to exist between the liquid limit and four other test constants, the plastic limit, the shrinkage limit, the centrifuge moisture equivalent, and the field moisture equivalent. In the paragraphs which follow the test constants are defined, their significance is explained, and the relations which form the basis of the soil identification chart are developed.

Critical moisture.—The deformations of either confined or unconfined soil samples under constant load increase with increase of moisture content at a consistent rate until a given moisture content known as the critical moisture is reached. When the moisture content is increased above this value the deformations of the samples increase at a very much greater rate than for similar moisture increases below the critical moisture. This fact is illustrated in Figure 37, which is the reproduction of a curve published previously in PUBLIC ROADS (*12*).

It will be noted in this figure that, for a load of 5.6 pounds per square inch, the deformation increases at the rate of about 0.00026 inch for each 1 per cent increase in moisture content below 26.7 per cent, the critical moisture. For increase in moisture content above 26.7 per cent the deformations increase at the rate of 0.022 inch for 1 per cent increase in moisture content.

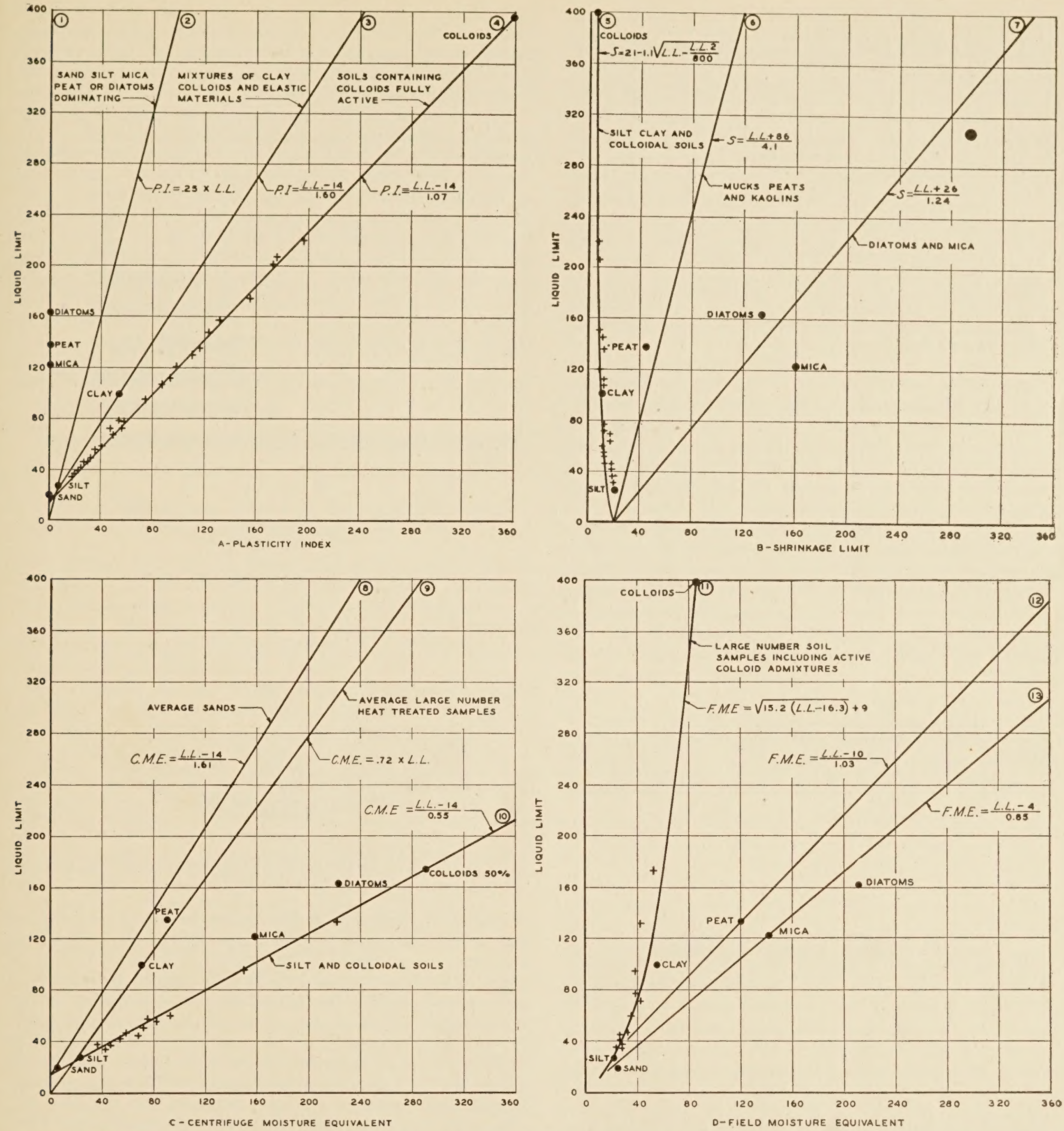


Figure 36.—The Soil Identification Chart: Relations Existing Between Liquid Limit and Other Test Constants

The ratio of applied load to corresponding deformations for different soils is not related to the critical moistures of these soils. As a consequence, therefore, the relation between the critical moisture of one soil and the critical moisture of another soil is not indicative of the relative supporting powers of the two soils. Furthermore, the degree of support indicated by the load-deformation relationship of soils at the critical moisture content varies widely in different soils.

The critical moisture is approximately equal to the plastic limit of cohesive soils and to 75 per cent of the liquid limit of cohesionless soils, which do not have plastic limits (*12*). The determination of the critical moisture is not a routine test. It assists in the explanation of the significance of the plasticity tests, and is of considerable practical importance, because it indicates the extent to which the moisture contents of soils must be reduced in order that they may be stabilized by drainage.

Plastic limit.—This constant is defined as the lowest moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil can be rolled into threads one-eighth inch in diameter without the threads breaking in pieces.

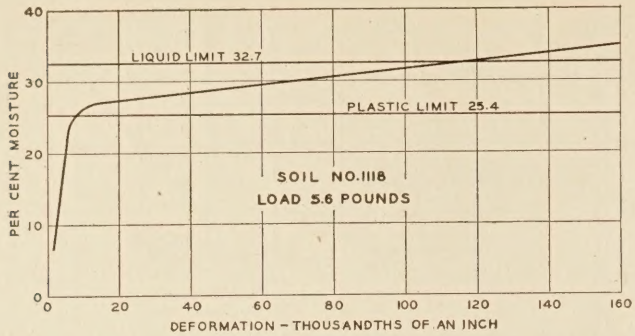


Figure 37.—Deformation of Soil Specimen with Increase in Moisture Content. Note Sudden Increase in Rate of Deformation as Critical Moisture Is Reached. Plastic Limit and Liquid Limit Are Also Marked on the Curve

Figure 38 shows the nature of the test for determination of the plastic limit. The upper sample, having a moisture content above the plastic limit, can be rolled into threads less than one-eighth inch in diameter without crumbling under the pressure exerted by the hand. The pressure required to deform the threads varies widely with the character of the soil. The lower part of the drawing shows a soil thread which has crumbled because the moisture content of the soil has been reduced by evaporation to the plastic limit or below.

The prime importance of the plastic limit with respect to this discussion is the fact that it furnishes part of the data required for computing the plasticity index. The following significant relationships should also be noted:

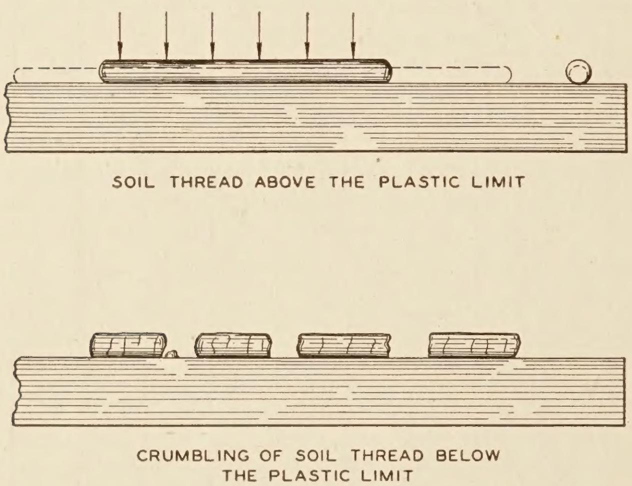


Figure 38.—Phenomenon Occurring During the Plastic Limit Test

The plastic limit (*a*) equals approximately the moisture content at which a minimum capillary pressure of 2.5 kilograms per square centimeter acts upon the soil sample; (*b*) equals the moisture content at which the coefficient of permeability of homogeneous clays becomes practically equal to zero; (*c*) equals the moisture content above which water evaporates about 4 per cent faster from a clay sample than from the free water surface; (*d*) equals the moisture content at which the speed of evaporation starts to decrease; (*e*) equals the moisture content below which the physical properties of water are no longer identical with those of free water (*12*).

Sand, mica, diatoms, and peat have no plasticity and therefore do not have plastic limits. Silts occasionally may have plastic limits. Clay and colloids have plastic limits. Of the representative subgrade soil constituents referred to in this report (p. 119), the silt has a plastic limit equal to 20, that of the clay is 46, and that of the colloids is 45.

Liquid limit.—The liquid limit is defined as that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil will just begin to flow when lightly jarred 10 times. According to this definition soils at the liquid limit have a very small but definite shear resistance, which may be overcome by the application of very little force. This resistance, it should be noted, is by definition practically constant for all soils when at the liquid limit, as contrasted with the variable support furnished by different soils when at the plastic limit. At the liquid limit the cohesion is practically equal to zero (*12*).

The nature of the liquid limit test is indicated in Figure 39. The soil sample is placed in a porcelain

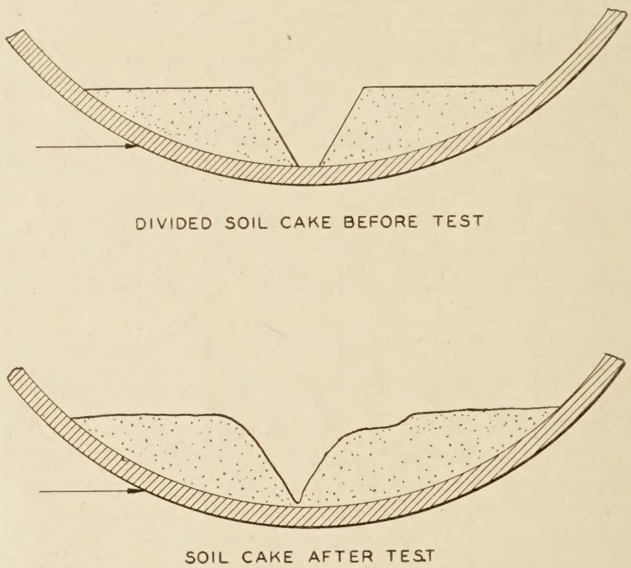


Figure 39.—Phenomenon Occurring During the Liquid Limit Test

evaporating dish about 4½ inches in diameter, shaped into a smooth layer about three-eighths inch thick at the center, and divided into two portions by means of a grooving tool. The dish is held firmly in one hand and tapped lightly 10 times against the palm of the other hand. If the lower edges of the two soil portions do not flow together as shown in the lower part of Figure 39, the moisture content is below the liquid limit. If they flow together before 10 blows have been struck, the moisture content is above the liquid limit. The test is repeated, with more or less moisture present, as the case may be, until a condition is reached where the two edges exactly meet after 10 blows have been struck. The arrows indicate the lateral flow, or shear failure, caused by the 10 blows.

The force created in the soil during the test is a function of the specific gravity of the soil particles combined with full or partial hydrostatic uplift, depending upon whether the soil is of the expansive or nonexpansive type.

Liquid limits of nonexpansive cohesionless soils indicate the moisture content required to lubricate the grain surfaces sufficiently to cause flow under the prescribed force. Liquid limits of expansive cohesionless soils indicate the degree of expansion required to reduce to practically zero the cohesion furnished by capillary pressure, skin friction, etc. Liquid limits of expansive cohesive soils indicate the degree of expansion required to reduce to practically zero the true cohesion of the soil particles in addition to that furnished by the capillary pressure, skin friction, etc.

In this way the liquid limit serves to distinguish (*a*) sands, with respect to the resistance which they furnish

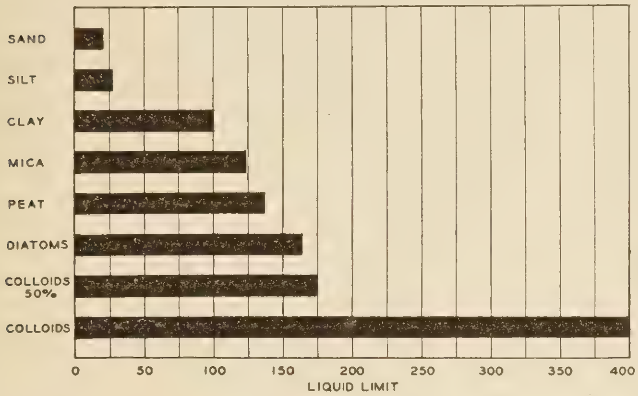


FIGURE 40.—LIQUID LIMITS OF REPRESENTATIVE SOIL CONSTITUENTS

to flowing, and (*b*) all other soils with respect to the relative volume of the pores of capillary dimension which these soils possess. The wide range in the liquid limits of different soils is illustrated by Figure 40, which show graphically the liquid limits of the representative soil constituents.

The liquid limits of the uniform subgrade groups may be stated approximately as follows:

Group A–1.—Generally greater than 14 and less than 25.

Group A–2.—Generally greater than 14 and less than 35.

Group A–3.—Varies in value from slightly less than 10 to slightly more than 35. Small liquid limits, such as 10 to 14, signify the beach and other rounded sand grains which, when sufficiently wetted, will slide over each other, i. e., flow, because of the lubrication of their surfaces. Grains uniform in size and perfectly spherical in form would probably flow without being lubricated. An abnormally large liquid limit, 30 to 35 in the case of sands, signifies high resistance to sliding furnished by a high degree of surface roughness or angularity of grain.

Group A–4. Generally greater than 20 and less than 40.

Groups A–5, A–6, and A–7.—Usually greater than 35.

Group A–8.—Likely to be greater than 45.

Plasticity index.—This term is defined as the difference between the liquid limit and the plastic limit. Plasticity indices equal to zero designate nonplastic soils, i. e., those which have no plastic limits.[8] At the plastic limit the soil particles may be considered as having acquired a degree of lubrication sufficiently high to permit them to slide over each other when loaded, although still possessing cohesion in appreciable amount. At the liquid limit, according to definition, the soil particles are separated to such an extent that practically no cohesion exists between them.

It follows, therefore, that the difference between the plastic limit and the liquid limit indicates the increase in moisture content required to increase the thickness of the water films separating the soil particles to a degree such that the cohesion existing between them is reduced practically to zero. Thus the plasticity index may be considered as a measure of the cohesion possessed by the soil.

As stated above, this cohesion consists of two parts: (*a*) That furnished by capillary pressure, skin friction, etc., and (*b*) that furnished by the true cohesion (molecular attraction) of the soil particles. To illustrate, assume that a pebble is first immersed and then removed from water. The adhesion existing between the surface of the pebble and the water particles in intimate contact with it is very high, but decreases rapidly in amount as the distance separating the water molecule from the pebble surface increases. Consequently the water molecules separated farthest from the pebble surface flow off under the force of gravity.

Two soil particles may be held together by both the adhesion possessed by water for the surface of the soil particles and the cohesion existing between water molecules. With increasing thickness of water film separating the two soil particles the attraction which tends to hold them together rapidly decreases to the vanishing point.

True cohesion existing between soil particles, like the adhesion existing between water molecules and the pebble surface, decreases rapidly in degree as the distance separating the soil particles increases. When the two soil particles possess true cohesion they are held together by a force exceeding that furnished by the molecular attraction of water. Consequently, the thickness of water film required to overcome the true cohesion existing between soil particles is greater than that required to overcome the adhesion due to capillary pressure, skin friction, etc.

If two glass plates are wetted with a small amount of water and pressed together they can not be pulled apart without the application of an appreciable external force. A very slight force, however, may serve to cause one plate to slide upon the other. This condition is analogous to the moisture content of the soil when at the plastic limit. By increasing the thickness of water film separating the two plates, one may be caused by only its own weight to separate from the other. This is analogous to the moisture content of the soil when at the liquid limit. The increase in thickness of water film required to produce the change from the sliding to the separated state represents that portion of the plasticity index required to equalize only the cohesion furnished by the molecular attraction existing between the water and the surfaces of the glass plates.

If, before being wetted and pressed together, the glass plates had been coated with a gluey colloid, the thickness of water film required to produce the change from the sliding to the separated state would of necessity have been larger than if the plates had not been so coated. The difference between the amount of water required to change the coated and the uncoated plates from the sliding to the separated state may be regarded as analogous to the amount of water required to overcome the cohesion possessed by the colloidal particles.

It was stated above that the critical moisture of cohesionless soils equals 75 per cent of the liquid limit. It follows that 25 per cent of the liquid limit may be assumed to indicate the amount of water required to overcome the cohesion furnished by the capillary pressure, skin friction, etc., in soils. If the same principle is applied to cohesive soils, it may be assumed that that portion of the plasticity index exceeding 25 per cent of the lower liquid limit is the amount of water required to overcome the true cohesion existing between the individual soil particles. Consequently, the quantity obtained by subtracting 25 per cent of the liquid limit

[8] Only a very few soils of the thousands tested in the subgrade laboratory of the Bureau of Public Roads have had plastic limits equal to the liquid limits.

from the plasticity index, indicates the relative amounts of true cohesion possessed by the surfaces of the soil particles. Thus the relation existing between the liquid limit and the plasticity index may serve very well to disclose certain characteristics of the soil.

Figure 36, A, illustrates relations existing between the plasticity index and the liquid limit. The large dots denote the relations given in Table 5 for representative soil constituents. Curve 1, indicating a plasticity index of zero, is characteristic of sand, mica, peat, and diatoms. Curve 2 represents the relation, plasticity index = 0.25 × liquid limit. This relation is characteristic of soils containing sand, silt, mica, peat, or diatoms in dominating amounts. Curve 3 indicates the statistical relationship given by tests of a very large number of samples of soils containing clay, colloids, and elastic materials such as peat or mica, and also soils containing colloids not fully active because of either heat treatment or state of flocculation. Curve 4 shows the relation given by compressible mixtures of fully active colloids. The crosses adjacent to curve 4 denote results given by tests of soil samples containing admixtures of bentonite, 1.5 to 50 per cent.

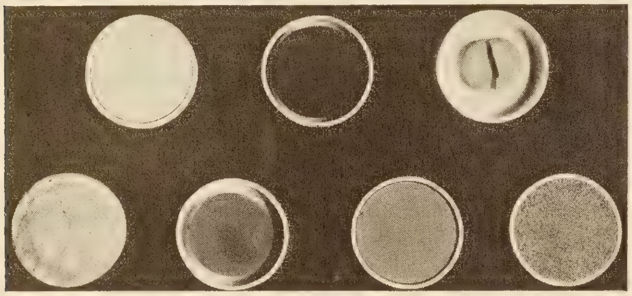


FIGURE 41.—SHRINKAGE OF REPRESENTATIVE SOIL CONSTITUENTS. TOP ROW, LEFT TO RIGHT, DIATOMS, PEAT, COLLOIDS; BOTTOM ROW, LEFT TO RIGHT, MICA, CLAY, SILT, SAND

The members of the uniform subgrade groups generally have plasticity indices as follows:

Group A–1.—Usually less than 8 and generally related to their liquid limits according to the relation shown by curve 2.

Group A–2.—Usually less than 15.

Group A–3.—Have no plasticity; therefore, do not have plasticity indices.

Group A–4.—Less than those indicated by curve 3.

Group A–5.—Seldom greater than those indicated by curve 3.

Group A–6.—Approximately equal to those indicated by curve 4.

Group A–7.—Generally greater than those indicated by curve 3 and smaller than those indicated by curve 4.

Group A–8.—Generally less than those indicated by curve 3.

Shrinkage limit and shrinkage ratio.—The mechanics of soil shrinkage was described in Part I of this report (PUBLIC ROADS, June, 1931, pp. 102 and 103) and will not be repeated here. The shrinkage limit is defined as the moisture content, expressed as a percentage of the dry weight, required to fill the pores of a soil sample which has been dried to constant weight from a moisture content sufficient in amount to fill the soil pores completely. Those soils which shrink during this drying process are referred to as possessing significant shrinkage limits. The shrinkage limits of nonexpansive soils such as sand, etc., which do not shrink during this drying process may be computed, as shown later, when the specific gravities of the soils are known. Such shrinkage limits are termed "theoretical" to distinguish them from the "significant" shrinkage limits. Sand and mica have theoretical and the other constituents have significant shrinkage limits. In general, only the significant limits are given in routine test reports.

Figure 41 illustrates the shrinkage of representative soil constituents. The pats shown were dried to constant weight from a moisture content slightly above the liquid limit. The container in which the pat rests represents the original volume of the pat.

The capillary pressure per square centimeter exerted upon the surface of the drying soil sample may be computed as follows:

Assume the voids in the soil mass to be of square cross section and to have each a width equal to a. The perimeter of each tube equals $4a$. Since the force exerted by capillary pressure is equal to 0.0764 grams per centimeter (*1*), the force exerted upon each tube, is given by the expression, $4\,a \times 0.0764 = 0.306a$.

If we should assume that the soil surface is completely covered by such openings, the number of openings per square centimeter equals the reciprocal of the area, a^2, of each opening. Consequently, when we designate $S.\ F.$ as the force causing shrinkage in a soil, we have

$$S.\ F. = \frac{0.306a}{a^2} = \frac{0.306}{a}. \quad \text{-------------(14)}$$

According to this relation, illustrated graphically in Figure 42, the pressure exerted upon a soil possessing voids 0.1 millimeter wide equals 30.6 grams per square centimeter and that exerted upon a soil possessing voids 0.001 millimeter wide equals 3,060 grams per square centimeter. This is the explanation of the fact, illustrated in Figures 41 and 43, that colloids upon drying compact the greatest amount, clay a less, and silt a still less amount.

The shrinkage ratio is defined as the volume change, expressed as a percentage of the volume of the dry soil cake, divided by the moisture loss above the shrinkage limit, expressed as a percentage of the weight of the dry cake.

A determination of skrinkage limit and shrinkage ratio is illustrated in Figure 44. As a soil pat consisting of Yaguajay clay was being dried from the wet state, the changes of both volume and moisture content were determined at different times. The results of these observations are plotted as small circles in the figure. The abscissa of any point on the curve denotes volume change expressed as a percentage of the volume of the dry pat; the ordinate denotes moisture content, expressed as a percentage of the weight of the dry pat.

These points, it will be noted, lie practically on a straight line which intersects the zero line at a moisture content of 11.1 per cent. Consequently no volume change will occurr in the soil cake when the moisture content is reduced from 11.1 to 0 per cent, and 11.1 per cent is the shrinkage limit.

As the moisture content was reduced from 107.8 to 11.1 per cent, the pat underwent a volume change equal to 200 per cent of its volume when dry. The shrinkage ratio is therefore obtained by the computation,

$$\frac{200}{107.8-11.1} = \frac{200}{96.7} = 2.07$$

The shrinkage limit, the shrinkage ratio, and the specific gravity are interrelated as follows:

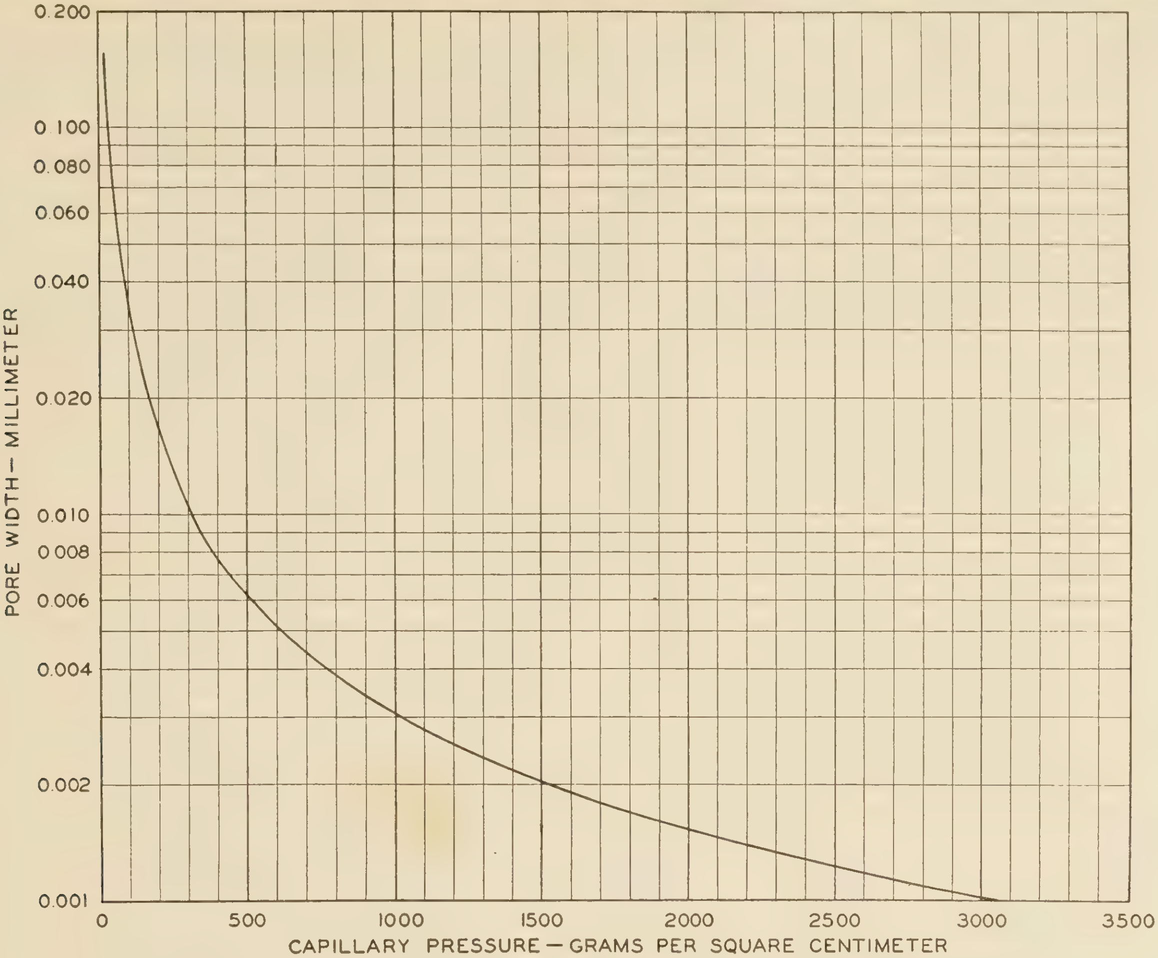


FIGURE 42.—THEORETICAL RELATION BETWEEN PORE WIDTH AND CAPILLARY PRESSURE

Let

S = shrinkage limit.
w = moisture content.
V = volume of wet soil cake.
V_o = volume of dry soil cake.
W = weight of wet soil cake.
W_o = weight of dry soil cake.
R = shrinkage ratio.
G = specific gravity of soil particles.

The total volume change, $V-V_o$, is equal to the moisture loss occurring between the original moisture content, w, and the shrinkage limit, S.

The weight of water lost between these two limits is given by the expression,

$$\frac{w \times W_o}{100} - \frac{SW_o}{100} = \frac{W_o\ (w-S)}{100}$$

If measurements are made in grams and centimeters,

$$\frac{W_o\ (w-S)}{100} = \text{volume of water lost.}$$

$$= V - V_o$$

$$W_o w - W_o S = (V - V_o) \times 100$$

$$W_o S = W_o w - (V - V_o) \times 100$$

$$S = w - \frac{V - V_o}{W_o} \times 100 \text{ ----------- } (15)$$

And

$$w - S = \frac{V - V_o}{W_o} \times 100$$

The shrinkage ratio, R, defined as the volume change in percentage of the dry volume, divided by the moisture loss above the shrinkage limit in percentage of the dry weight, is given by the equation,

$$R = \frac{\frac{V - V_o}{V_o} \times 100}{w - S}$$

Substituting the value of $w-S$ previously obtained, we have

$$R = \frac{\frac{V - V_o}{V_o} \times 100}{\frac{V - V_o}{W_o} \times 100} = \frac{W_o}{V_o} \text{ ----------- } (16)$$

The specific gravity equals the weight of the dry soil in grams divided by its true volume in cubic centimeters. The true volume of the dry soil equals the apparent volume, V_o, minus the water content at the shrinkage limit, $\frac{SW_o}{100}$. We have, therefore,

$$G = \frac{W_o}{V_o - \frac{SW_o}{100}} = \frac{1}{\frac{V_o}{W_o} - \frac{S}{100}} = \frac{1}{\frac{1}{R} - \frac{S}{100}} \text{ ----------------- } (17)$$

At one determination of the volume of the Cuban (Yaguajay) soil the weight of the wet pat, W, was 43.74 grams and its volume, V, was 28.04 cubic centimeters. In the dried state the weight of the pat, W_o, was 25.08

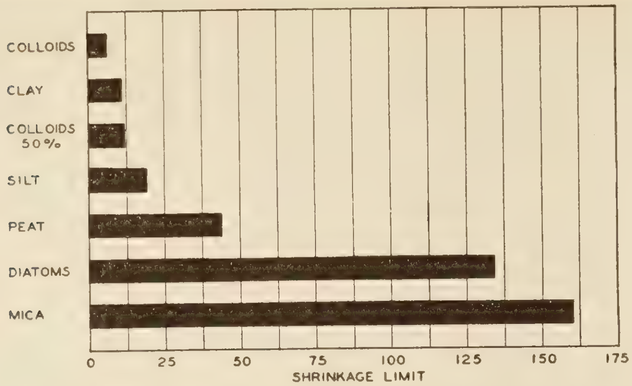


Figure 43.—Shrinkage Limits of Representative Soil Constituents

grams and its volume, V_o, was 12.16 cubic centimeters. The value of the moisture content, w, of the wet soil is given by the expression

$$w=\frac{W-W_o}{W_o}\times 100=\frac{43.74-25.08}{25.08}\times 100=74.40 \text{ per cent.}$$

And the shrinkage limit,

$$S=w-\frac{V-V_o}{W_o}\times 100=74.40-\frac{28.04-12.16}{25.08}\times 100=$$

11.1 per cent.

This value checks with that shown in Figure 44.

The volume change which occurred during the loss of 74.4 per cent of moisture, is given by the equation

$$C_o=\frac{V-V_o}{V_o}\times 100=\frac{28.04-12.16}{12.16}\times 100=130.6 \text{ per cent.}$$

This value is shown in Figure 44 by means of a double circle.

The shrinkage ratio $R=\frac{W}{V_o}=\frac{25.08}{12.16}=2.06$ as compared with 2.07, the value obtained from Figure 44.

It will be noted that the shrinkage ratio R is also the apparent specific gravity of the soil. Thus the weight of a cubic foot of the Cuban clay in the dried state equals $62.5\times 2.06=128.75$ pounds.

The specific gravity computation may be subject to appreciable error, since it is based on the assumption that the relation which exists between moisture loss and percentage volume change is as consistent as that indicated in Figure 44. Certain elastic soils are likely to expand during the period when the moisture content is being reduced from the shrinkage limit to zero. Mica is one of these materials and the degree to which it may expand under these conditions is illustrated in Figure 45. This accounts in part for the high shrinkage limit given for mica in Figure 43. Furthermore, the presence of air in the pores of a drying soil may prevent it from shrinking exactly in accordance with the relation shown in Figure 44. Figure 46 shows how different soils may vary in drying from the relation as stated. Table 6 illustrates the variation which may exist between actual and computed specific gravities of soils.

Table 6.—*Comparison of specific gravities as obtained by actual determination and by computation from formula 17*

Soil No.	Specific gravity, actual determination	Specific gravity computed from formula	Soil No.	Specific gravity, actual determination	Specific gravity computed from formula
2, 262	2. 72	2. 70	2, 242	2. 640	2. 67
2, 264	2. 70	2. 75	2, 499	2. 815	2. 80
2, 280	2. 59	2. 55	2, 443	2. 752	2. 77
2, 235	2. 50	2. 38	1, 688	2. 716	2. 72
2, 278	2. 64	2. 58	1, 606	2. 740	2. 75
2, 261	2. 62	2. 72	1, 547	2. 653	2. 67
2, 272	2. 65	2. 71	1, 611	2. 612	2. 62
2, 243	2. 71	2. 79	2, 370	2. 715	2. 72
2, 251	2. 54	2. 62	2, 368	2. 660	2. 66
2, 248	2. 63	2. 65	2, 365	2. 640	2. 51
2, 250	2. 71	2. 67			

Figure 36, B illustrates relations existing between the shrinkage limit and the liquid limit. The large dots denote the relations given in Table 5 for the representative soil constituents. Curve 5 is characteristic

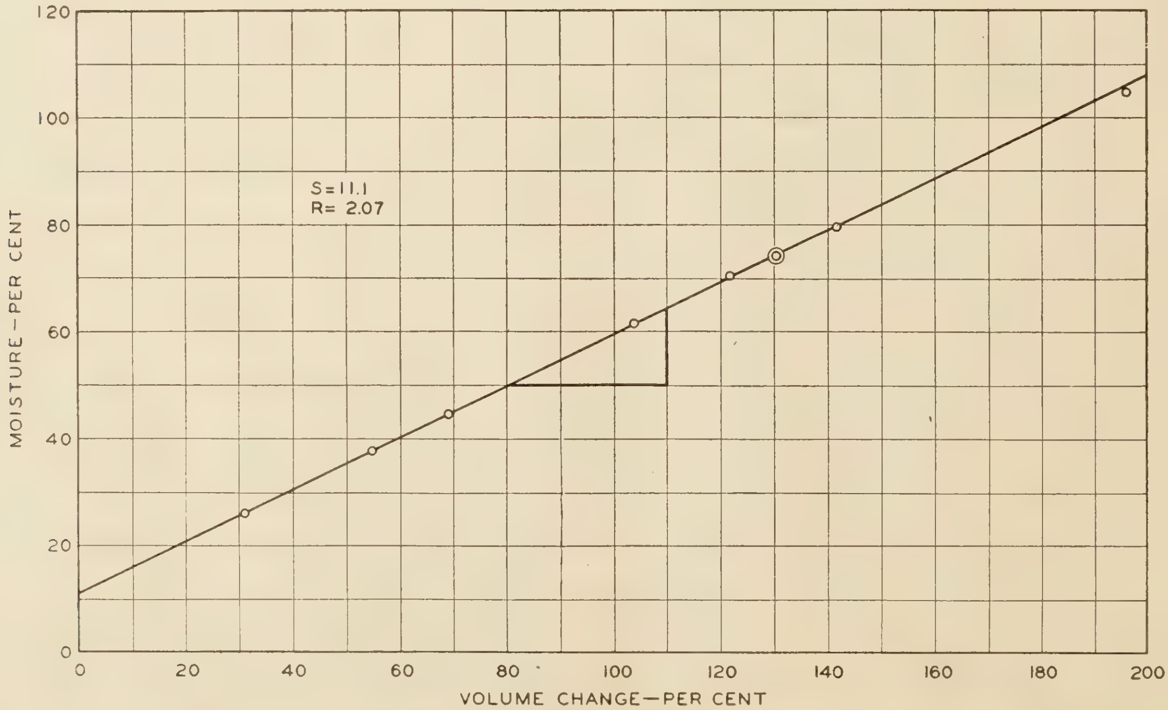


Figure 44.—Relation Between Moisture Content and Volume Change for Yaguajay Clay

FIGURE 45.—SIDE VIEW OF MICA PAT SHOWING EXPANSION ON DRYING TO CONSTANT WEIGHT FROM A MOISTURE CONTENT SLIGHTLY ABOVE THE LIQUID LIMIT. THE MICA JUST FILLED THE CUP WHEN WET

of silt, clay, and colloidal soils. It also indicates approximately the average relation given by tests of a large number of natural soil samples (*21*). The small crosses adjacent to this curve represent results of tests performed on natural soil containing bentonite in admixtures varying between 1.5 and 50 per cent. Curve 6 indicates the moderately high shrinkage limits characteristic of mucks, peats, and kaolins. Curve 7 represents diatoms and mica, which have high shrinkage limits.

Shrinkage limits characteristic of the uniform subgrade groups are as follows:

Group A–1.—Generally greater than 14 and less than 20.

Group A–2.—May be either theoretical or significant depending upon other constants. Not likely to exceed 25 when significant.

Group A–3.—No significant shrinkage limit.

Group A–4.—Generally less than 25. Increase in expansive properties of members of this group indicated when shrinkage limits exceed 20 and approaches relationship represented by curve 6.

Group A–5.—Generally greater than 30 and greater than 50 for very undesirable members of this group. May approach relation indicated by curve 6 for silts containing peat and approach relation indicated by curve 7 for soils containing either diatoms or mica in appreciable amount.

Group A–6.—Not likely to exceed in appreciable amount values represented by curve 5.

Group A–7.—For flocculated varieties of this group, may slightly exceed values given by curve 5. For varieties containing mica, peat or diatoms, may exceed very appreciably values indicated by curve 5, but generally less than those indicated by curve 6. Soils of this group subject to frost heave have the higher shrinkage limits.

Group A–8.—Generally in neighborhood of values indicated by curve 6, and seldom greatly in excess of those values.

Centrifuge moisture equivalent.—This constant is defined as the moisture content, expressed as a percentage of the weight of the oven-dried soil, retained by a soil sample after first being soaked in water for 6 hours, then drained in a humidifier for 12 hours, and finally centrifuged under an acceleration of 1000 × gravity for 1 hour.

The action which takes place is illustrated in Figure 47. Water is forced outward through the bottom of the cup under the influence of two forces, the centrifugal force acting on the water and the pressure which the soil particles exert on one another. The centrifugal force acting on the water is proportional to the distance from the surface of the sample. Since the acceleration is $1000g$ the pressure at a distance y will be equal to y kilograms per square centimeter. The pressure which the individual particles exert upon each other in the direction of the axis of the cup is a function of the specific gravity of the particles, the distance from the surface, and the extent of hydrostatic uplift. If the

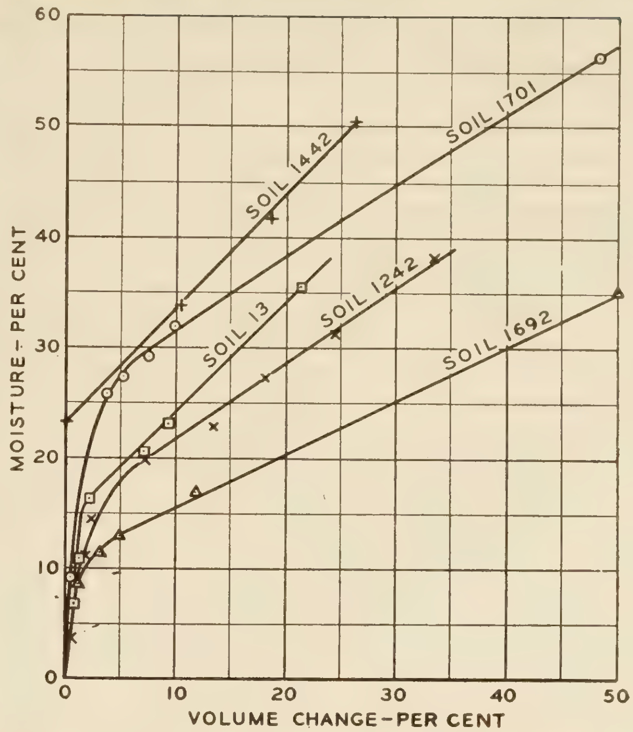


FIGURE 46.—TYPICAL CURVES SHOWING VARIATION OF VOLUME CHANGE WITH MOISTURE CONTENT

soil particles are not surrounded by water, the hydrostatic uplift is negligible, and we may define the pressure on the particles as Py kilograms per square centimeter, where P is a function of the specific gravity. This is the case with nonexpansive soils. In the case of expansive soils the hydrostatic uplift is much greater. If we assume it as full, the pressure becomes $(P-1)y$, or, at the bottom of the cup, $(P-1)h$.

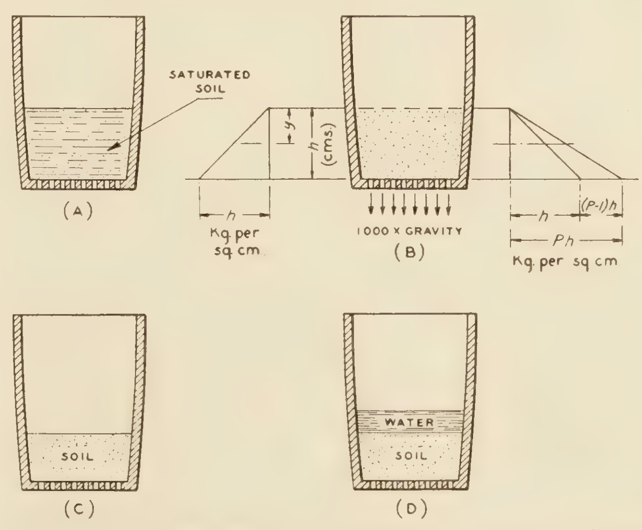


FIGURE 47.—PHENOMENON OCCURRING DURING THE CENTRIFUGE MOISTURE EQUIVALENT TEST

This combination of forces results in pressures which are equivalent to an average pressure on the sample of about 2 kilograms per square centimeter. As the intimacy of contact between soil particles is increased, the tendency is for water to be squeezed from the sample at both top and bottom. The tendency for water to be forced to the top is resisted by the centrifugal force acting on the water. This force is in turn opposed by the frictional resistance to flow offered by the surfaces of the soil pores and the capillarity of the soil, both of which increase as the soil mass decreases in volume during the test. If the resisting force is greater than the centrifugal force water remains at the top of the sample, producing the condition called waterlogging. The amount of water which is forced through the sample and escapes through the outlets at the bottom of the cup is thus dependent on the permeability of the soil.

Thus the centrifuge moisture equivalent (*a*) serves to distinguish soils which are permeable (sand, silt, mica, diatoms, peat or flocculated clay dominating) from those which are impermeable (clay and colloids dominating) when compressed by a centrifugal force equal to about 2 kilograms per square centimeter (*12*); (*b*) serves to disclose to some extent the degree of capillarity possessed by permeable soils; (*c*) furnishes a means of distinguishing permeable soils of the nonexpansive from those of the expansive varieties.

The moisture equivalents of permeable soils, for instance, decrease consistently when the sand content of the soils is increased. The shrinkage limits of silt and clay soils having capillarity in appreciable amount increase at a very slow rate with increase in the sand content of the soils until the amount of sand added be-

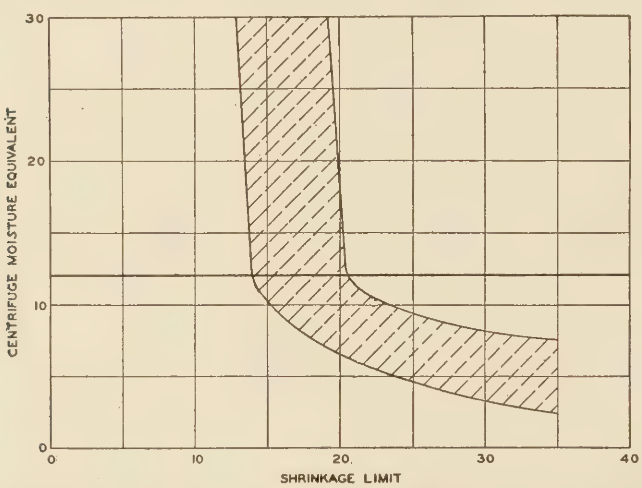


Figure 48.—Relation Between Centrifuge Moisture Equivalent and Shrinkage Limit for Sand Admixtures

comes sufficient to reduce the capillary properties of the soil very appreciably. At this sand content the shrinkage limits of the soils become theoretical instead of significant and increase at an abnormally high rate with further additions of sand. Therefore, by plotting the centrifuge moisture equivalents against the corresponding shrinkage limits of soils to which sand has been added in increasing amounts, the centrifuge moisture equivalent value at which the shrinkage limits suddenly begin to increase is easily determined. This value of the centrifuge moisture equivalent should indicate the degree of capillarity below which expansion and shrinkage become negligible in amount.

Figure 48 shows the results furnished by a determination of this character. The soil constants were obtained from tests performed upon a number of soils containing sand in amounts varying between 20 and 80 per cent. The results, it will be noted, are grouped in a well defined band which indicates a very pronounced increase in the theoretical shrinkage limits when the amount of contained sand is sufficient to reduce the centrifuge moisture equivalents to less than 12.

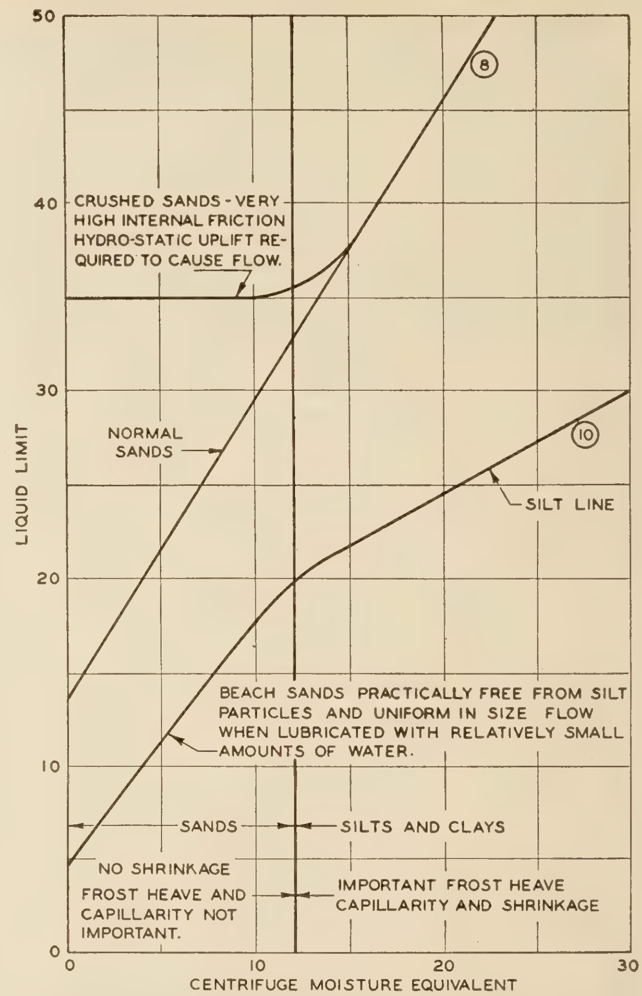


Figure 49.—Relation Between Liquid Limit and Centrifuge Moisture Equivalent for Different Types of Sand

Since detrimental frost heave occurs only in soils having capillarity in appreciable amount, it should not occur in soils which have centrifuge moisture equivalents less than about 12. And this seems to be verified by the subgrade surveys of the bureau now in progress. To provide a proper factor of safety, however, materials used for porous (*7*) base courses in practice, should have centrifuge moisture equivalents not exceeding 6 or 8.

The centrifuge moisture equivalents of the representative soil constituents (Table 5) are shown graphically in Figure 36, C. This figure contains also curves 8, 9, and 10, which indicate important relations existing between the centrifuge moisture equivalent and the liquid limit. Curve 8 represents the relation between the liquid limits and the centrifuge moisture equivalents of average sands (absence of hydrostatic uplift). Curve 9 represents the statistical relation between the average liquid limits and centrifuge moisture equivalents obtained from a great number of heat-treated soil samples (*21*). Curve 10 represents the relation between the liquid limits and the centrifuge moisture equivalents of average silt and colloidal soils (particles acting under full hydrostatic uplift).

Members of the uniform subgrade groups may have centrifuge moisture equivalents as follows;

Group A–1.—Seldom appreciably greater than 15.
Group A–2.—Not likely to exceed 25.

Group A–3.—Not likely to exceed 12. In combination with the liquid limit, discloses the relative resistance to flowing possessed by sands equal in degree of capillarity. (See Fig. 49).

Group A–4.—Generally greater than 12, approaching values indicated by curve 10, but not likely to waterlog, although exceptions occur. When greater than the liquid limits in the absence of waterlogging, indicates especially unstable silts.

Group A–5.—Greater than 12 and not likely to waterlog, although exceptions occur. Often has values between curves 9 and 10. May approach those indicated by curve 8 for sand-mica mixtures.

Group A–6.—May approach values indicated by curve 10 for the highly colloidal soils, with values lying between curves 9 and 10 for clay soils containing sand in appreciable amount. Likely to waterlog when exceeding 40.

Group A–7.—Generally greater than values indicated by curve 9 and less than values indicated by curve 10. May not waterlog with centrifuge moisture equivalents as high as 90.

Group A–8.—Generally between curves 9 and 10.

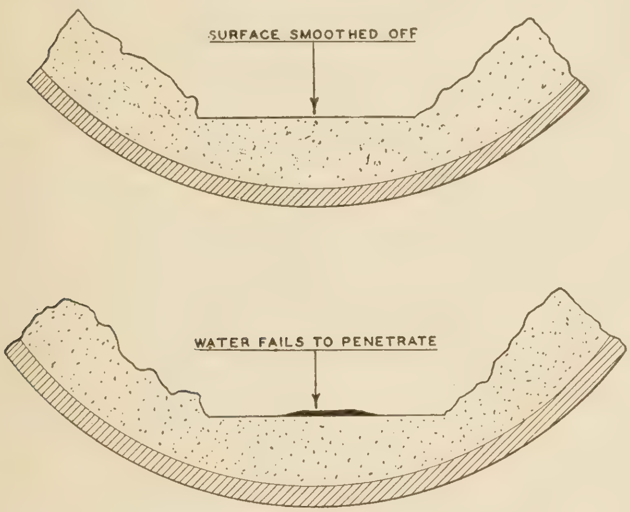


FIGURE 50.—PHENOMENON OCCURRING DURING THE FIELD MOISTURE EQUIVALENT TEST

It is interesting to note also that adding sand to the silt and clay mixes, which are represented by curve 10, will change these soils to graded mixes of the Group A–1 or Group A–2 variety and the test relationship will approach that indicated by curve 9. Adding both sand and mica to a silt may cause its centrifuge moisture equivalent to assume the relation to the liquid limit represented by curve 9.

Field moisture equivalent.—This term is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed, but will instead spread out over the surface and give it a shiny appearance. (See fig. 50.)

The drop of water fails to penetrate the wet and smoothed soil sample (*a*) when the pores of nonexpansive soils (sands) are completely filled, (*b*) when the capillarity of cohesionless expansive soils (diatoms and mica) is completely satisfied, and (*c*) when cohesive soils possess moisture in amount sufficient to cause the smoothed surface of the sample to become impervious. This impervious skin may occur at moisture contents far below those required to satisfy the capillarity of cohesive soils.

That the moisture content at which the impervious skin is formed measures a definite soil property and is not dependent upon the time during which the soil remains wetted is evidenced by the fact that highly colloidal clays, whether wetted for several minutes or for 24 hours, generally resist the penetration of the drop of water at practically equal moisture contents.

Figure 36, D contains curves showing relations existing between the liquid limit and the field moisture equivalent. Curve 11 represents the statistical relationship which was found to exist between the averages of results furnished by tests performed upon a large number of natural soil samples (*21*). This curve, it will be noted, represents also the relation given by results of tests performed upon the soils containing admixtures of active colloids.

The positions of curves 12 and 13 were chosen arbitrarily to represent high and very high field moisture equivalents. The field moisture equivalents of the representative soil constituents (Table 5) are also shown graphically in Figure 36, D. The field moisture equivalent individually and in its relation to the other constants serves to furnish the following supplementary information with respect to the identification of subgrade soils.

Group A–1.—Field moisture equivalent not significant.

Group A–2.—Field moisture equivalent not significant.

Group A–3.—Field moisture equivalent indicates the porosity of these cohesionless materials when completely saturated; in combination with the liquid limit discloses the degree of saturation required to cause sands to have a very small shear resistance.

Group A–4.—When approximately equal to or larger than centrifuge moisture equivalents the field moisture equivalents indicate presence of expansive properties in detrimental amounts.

Group A–5.—Field moisture equivalents may approach values indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic silts containing mica or diatoms in appreciable amount. May not exceed those indicated by curve 11 for kaolins possessing good binder properties.

Group A–6.—May approach values indicated by curve 11 generally, but smaller when the grading of the colloidal clay soils of this group is such as to cause smoothed surface of soil when wetted to become highly impermeable.

Group A–7.—Soils of this group either flocculated or containing organic matter partially decomposed into the colloidal state

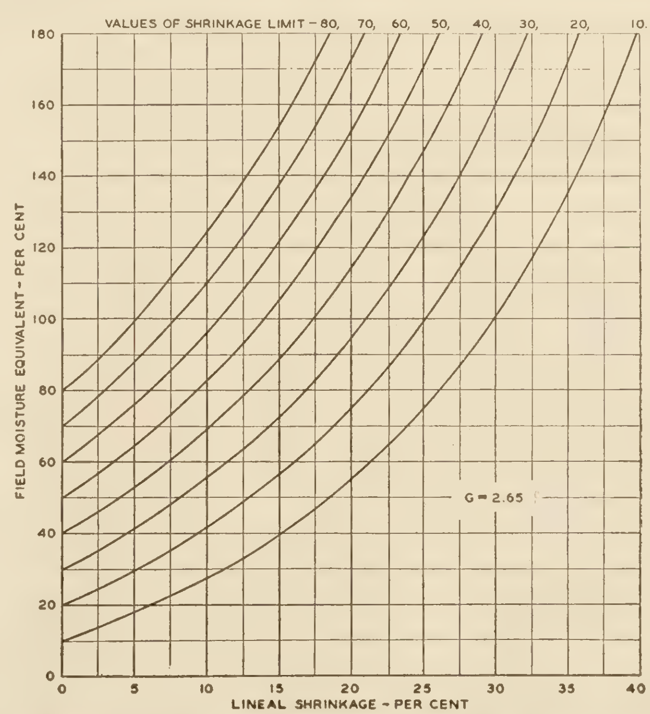


FIGURE 51.—CHART FOR ESTIMATING SHRINKAGE LIMIT FROM VALUES OF FIELD MOISTURE EQUIVALENT AND LINEAL SHRINKAGE

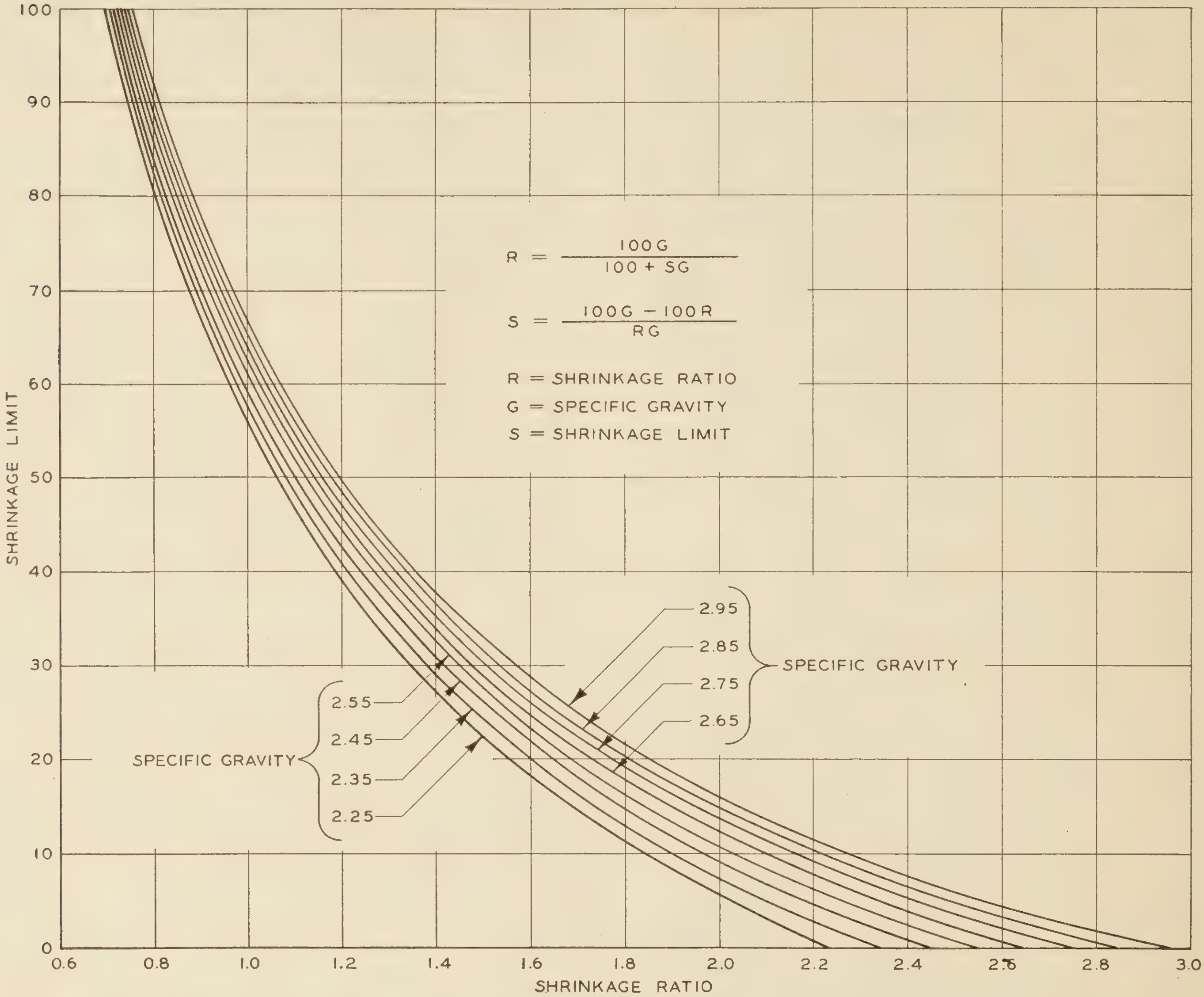


FIGURE 52.—RELATION BETWEEN SHRINKAGE LIMIT AND SHRINKAGE RATIO FOR DIFFERENT VALUES OF SPECIFIC GRAVITY

may have, as in the case of the Group A–6 soils, shrinkage limits approaching those represented by curve 5. Their field moisture equivalents, however, are likely to be appreciably greater than those indicated by curve 11.

Group A–8.—Same as group A–5.

Volumetric change.—Only in certain instances does the degree to which soils may shrink when dried out from an arbitrary wet state furnish information with respect to the identification of subgrade soils supplementary to that furnished by the constants discussed above. Volumetric changes from the field moisture equivalent are now computed only to determine whether those of graded materials and silty clays are larger or smaller than about 17. This limit, according to computation, is equivalent to a lineal shrinkage of 5, a value which has been established by A. C. Rose and C. H. McKesson (PUBLIC ROADS, August, 1924, September, 1925, and September, 1927), as representing the maximum degree of shrinkage properties which may be possessed by good soil mortars or stable subgrade soils. Thus, in certain instances the volumetric change assists in distinguishing the Group A–6 and Group A–7 soils which are inclined to shrink in appreciable amount (volumetric change approximately equal to or greater than 17) from the Group A–2, A–4, or A–5 varieties in which shrinkage is not important.

Let

F. M. E. = field moisture equivalent,
G = specific gravity of soil particles,
S = shrinkage limit,
R = shrinkage ratio.

The volumetric change, C_f, is given by the formula

$$C_f = (F.\ M.\ E. - S) \times R \quad \text{..........} (18)$$

We have

$$G = \frac{1}{\frac{1}{R} - \frac{S}{100}} \quad \text{..........} (17)$$

Hence,

$$R = \frac{100G}{100 + GS} = \frac{1}{\frac{1}{G} + \frac{S}{100}};$$

And, by substitution,

$$C_f = \frac{F.\ M.\ E. - S}{\frac{1}{G} + \frac{S}{100}} \quad \text{..........} (19)$$

No computation of the volumetric change is required for the average clay soils of the Group A–6 or Group A–7 variety when the shrinkage limits do not exceed those shown by curve 5 and the field moisture equivalents are not less than those indicated by curve 11. With a liquid limit of 35, for instance, such soils have shrinkage limits not exceeding 15 and field moisture equivalents not less than 26. By substitution of these values in equation 18, we find that the volumetric change, with an average R equal to 1.8, exceeds 20, and for this value the lineal shrinkage is larger than 5.

Lineal shrinkage.—The volumetric change, C_f, is expressed as a percentage of the dry volume of the soil cake. The lineal shrinkage, $L.\ S.$, expressed as a percentage of the length of wet soil bar, is defined by the formula

$$L.\ S.=100\left[1-\sqrt[3]{\frac{100}{C_f+100}}\right] \text{- - - - - - - - - - - - -} (20)$$

The lineal shrinkage, as such, possesses no more significance than the volumetric change. However, the lineal shrinkage combined with the field moisture equivalent offers a means of estimating the shrinkage limit which may be used when the first two values are known and the shrinkage limit has not been determined by test.

The relation between lineal shrinkage, field moisture equivalent, and shrinkage limit computed by equations 19 and 20 for soils having a specific gravity of 2.65 is shown graphically in Figure 51. According to this figure a soil having a lineal shrinkage of 17 combined with a field moisture equivalent of 82 would have a shrinkage limit of approximately 30.

The relation between shrinkage limit and shrinkage ratio for specific gravities varying from 2.25 to 2.95 is shown in Figure 52.

TEST CONSTANTS AND MECHANICAL ANALYSES STATISTICALLY RELATED

In addition to knowing the interrelationships between the test constants it may prove helpful also to have some conception of the average relations existing between the clay, silt, and sand contents of soils and their test constants.

The relations represented by curves 3, 5, 9, and 11 (fig. 36) are, in general, those which have been reported previously as "statistical" relationships between the averages of large numbers of individual soil tests (*21*). The average mechanical analysis of soils whose tests are related according to these statistical laws is shown in Figure 53 as a function of the liquid limit.

Both the grading represented in Figure 53 and the constants represented by curves 3, 5, 9, and 11 may be considered as characteristic of "average soils," and this fact may serve as a basis for estimating the relative degree to which particular soils possess certain characteristics.

An "average" or "statistical" soil containing 72 per cent clay and no sand has, according to Figures 53 and 36, constants as follows: Liquid limit, 100; plasticity index, 54; shrinkage limit, 11; centrifuge moisture equivalent, 72; and field moisture equivalent, 45. If a soil being investigated contains 72 per cent of clay and no sand, but the constants have the values, liquid limit, 50, plasticity index, 30, shrinkage limit, 15, centrifuge moisture equivalent, 60, and field moisture equivalent, 55, its constants may be expressed as ratios of the constants possessed by the statistical soils.

As the investigations in progress disclose more and more the soil constituents which cause the constants of

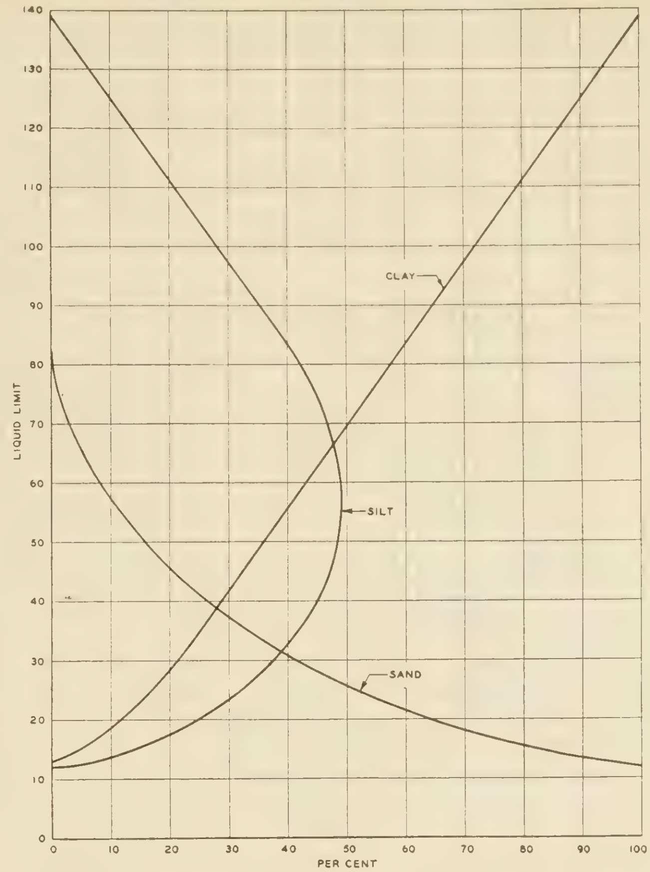


FIGURE 53.—AVERAGE MECHANICAL ANALYSIS OF SOILS WHOSE TESTS ARE RELATED ACCORDING TO CURVES 3, 5, 9, AND 11, FIGURE 36

various soils to differ from the constants of statistical soils, the relations discussed above will serve to reveal more accurately the constituents of which the soils being investigated are composed.

It is interesting to note that the representative soil constituent, clay, in Table 5, is for all practical purposes an "average" soil.

The constants of statistical soils containing both clay and sand in amounts equal to those indicated in Figure 53 are obtained in the same manner as the constants of statistical soils containing clay and no sand.

Thus the constants of a statistical soil containing 36 per cent clay and 16 per cent sand have the following values: Liquid limit, 50; plasticity index, 23; shrinkage limit, 13; centrifuge moisture equivalent, 36; and field moisture equivalent, 32.

It may prove helpful also to be able to estimate what might be termed the average influence exerted by sand admixtures upon the constants possessed by soils. The conversion curves of Figures 54 and 55 furnish a means of making estimates of this character. These curves are based on tests performed upon soils containing sand admixtures in different amounts. The sand content, referred to in these figures as "per cent sand," is expressed as a percentage of the combined weights of both sand admixture and soil.

Estimates furnished by means of conversion curves decrease in accuracy as the amount of the assumed sand admixture is increased. This is due to the fact that the curves indicate the average influence exerted by a number of sands and not the influence exerted by a particular sand. As admixtures of a particular sand

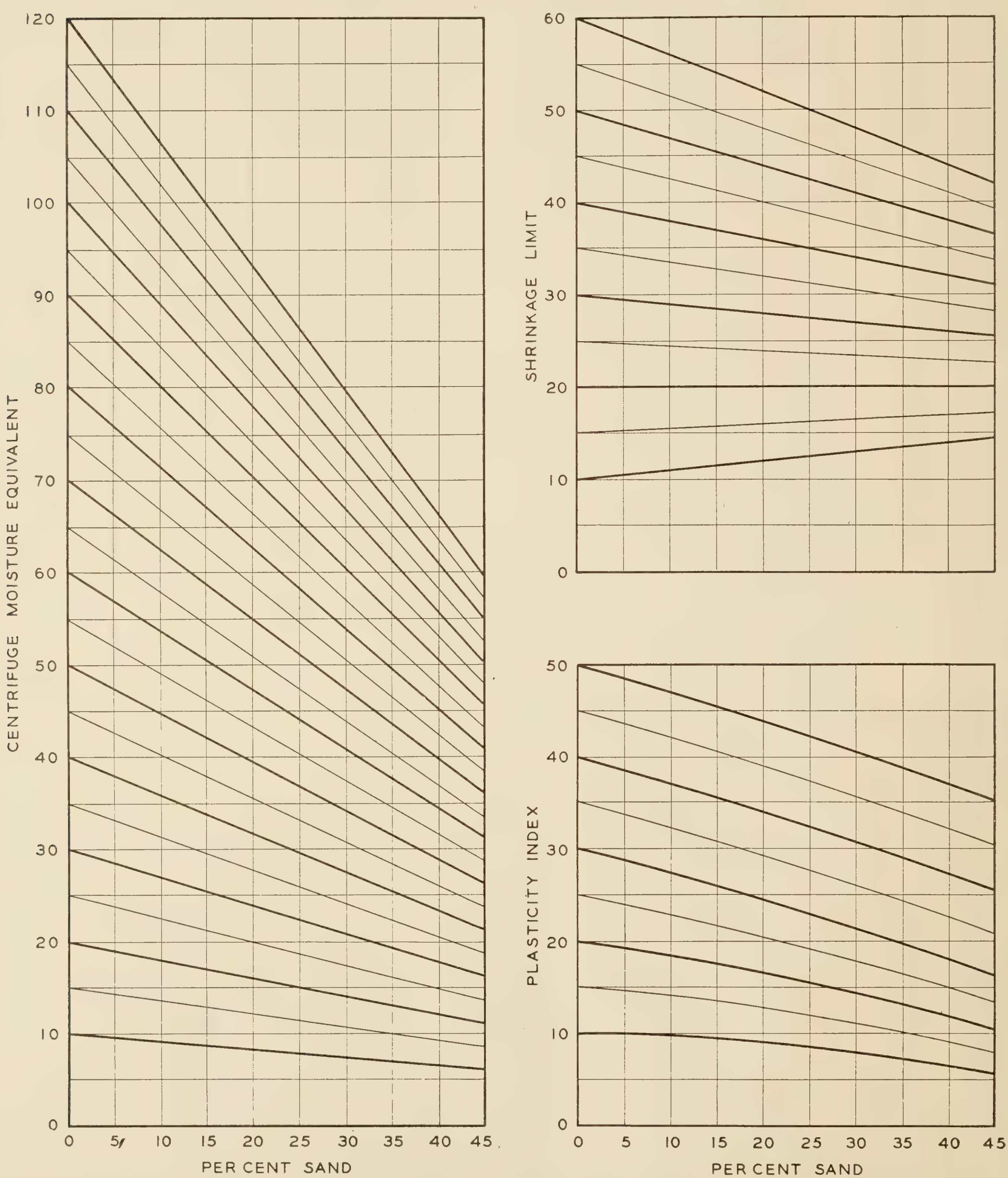


Figure 54.—Conversion Charts for Determining Effect of Sand Admixtures on Soil Test Constants

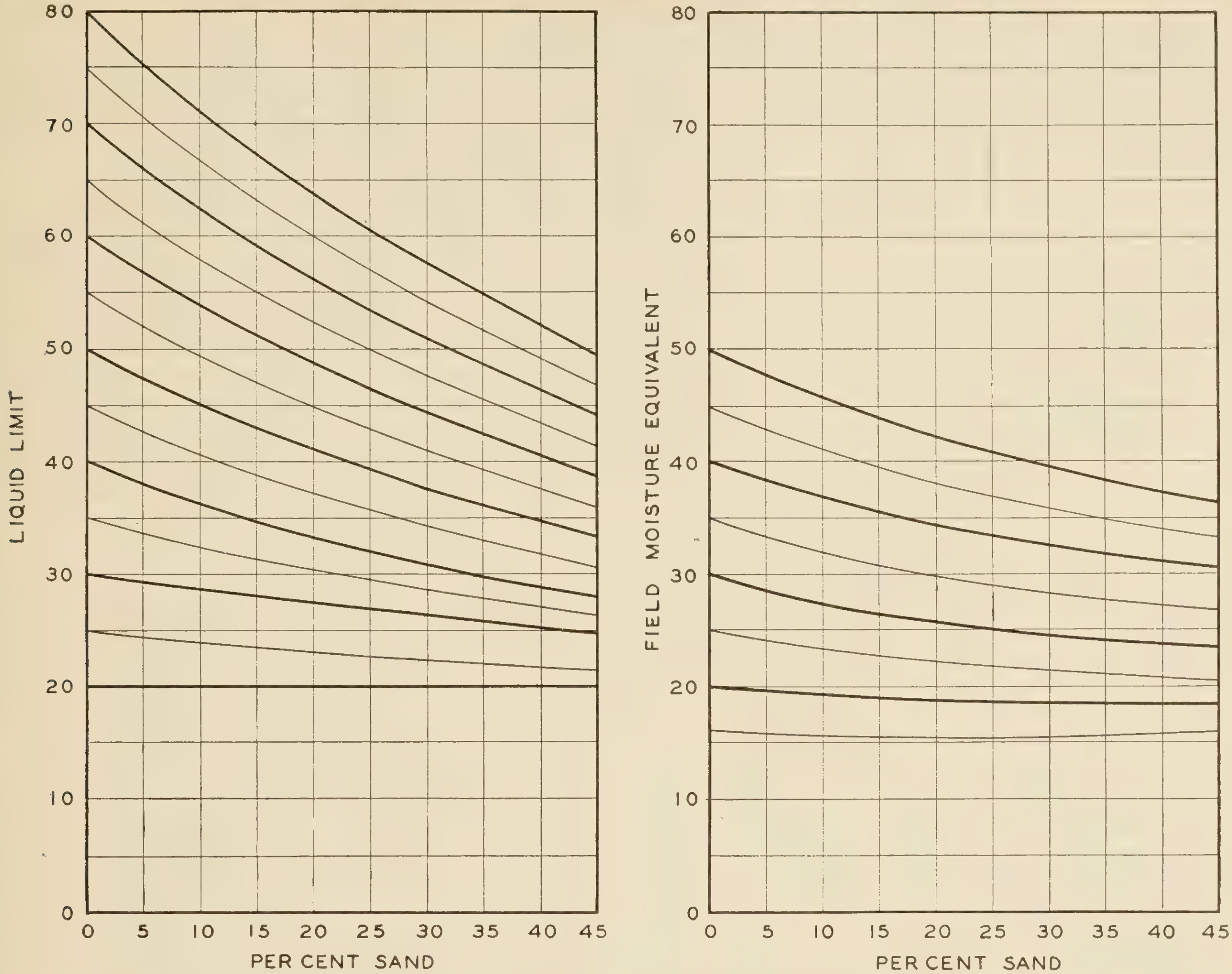


FIGURE 55.—CONVERSION CHARTS FOR DETERMINING EFFECT OF SAND ADMIXTURES ON SOIL TEST CONSTANTS

increase in amount, its individual characteristics, which depend on its grading, the size and shape of its grains, etc., will exert an increasingly important influence upon the test constants of the mixture, and cause them to vary from the estimates furnished by the conversion curves. This is especially true when the sand admixture increases in amount above about 45 per cent. As the sand admixture decreases in amount below about 45 per cent, the physical characteristics of the sand exert less and less influence on the soil test results.

In order to illustrate the use of Figures 54 and 55 let us assume that a soil sample has constants equal to those designated for sample A, Table 7. It is desired to estimate the influence exerted by adding sand in amount equal to 25 per cent of the weight of the soil. The assumed admixture in this case equals $\frac{25\times 100}{125}$ per cent or 20 per cent of the resulting combination of sand and soil. Consequently, we reduce each constant of sample A by an amount indicated in the corresponding conversion diagram for a sand admixture of 20 per cent. The resulting constants are given opposite sample X in Table 7.

TABLE 7.—*Test constants estimated on the basis of the conversion curves for sand admixtures*

Sample	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
A	45	19	20	1.7	33	30
X	37	16	20	[1] 1.7	26	26

[1] Specific gravity assumed to equal that of sample A. Therefore, with equal shrinkage limits the shrinkage ratios will be equal.

PART III: UTILIZATION OF THE SUBGRADE SOIL IDENTIFICATION CHART

The soil identification chart, shown in revised form in Figure 56 serves a triple purpose. First, it offers a means of identifying those subgrade soils whose performance in service has been learned; second, it assists in predicting the performance of soils comprising the subgrades of roads to be constructed; and, third, it assists in disclosing the influence exerted by either physical or chemical admixtures upon the performance of subgrade soils.

In order to demonstrate how these purposes are accomplished the improvement of soils by means of admixtures is illustrated and a limited number of soils belonging to the different groups are analyzed with respect to their constants. To facilitate this demonstration the basic requirements of good sand clay roads are discussed and both the grading and the constants indicative of soils of the different groups are reviewed.

GRADING OF GOOD TOP SOILS DEPENDS UPON THE CHARACTER OF THE BINDER

Theoretically stable mixtures consist of a well graded coarse material (grains larger than about 0.05 millimeter in diameter) possessing high internal friction, and a binder. The binder, which may be visualized as occupying the sand pores, should have sufficient cohesion to cement the sand grains together; and, upon wetting, the binder should expand in amount just sufficient to close the surface pores and thus prevent water from penetrating and softening the interior of the road surface. When the binder expands an amount greater than that required to close the sand pores, the sand grains are likely to become unseated, thus reducing the stability of the mixture. When the binder does not expand sufficiently to close the sand pores, water may enter and soften the road surface. It follows that the amount of binder required to furnish stable mixtures depends upon the expansion properties of the binder. Binders which are only slightly expansive may be used in an amount sufficient to fill the pores of the sand almost completely. As the expansive properties of the binder become more and more important, the amount used without danger of unseating the sand grains must of necessity become smaller and smaller.

Of two soils whose tendency to shrink or expand are equal, the one having the greater amount of cohesion should be the better binder. Of two soils having equal cohesion, that having the less tendency to shrink or expand should be the better binder, since a greater amount of it can be used than of the more expansive soil.

This theoretical conception that the amount of binder required depends upon the characteristics of the binder is substantiated by Doctor Strahan's (*22*) studies of roads in service. He emphasizes the fact that while soil mixtures having particular gradings are likely to produce stable wearing courses, the gradings are a qualitative rather than a quantitative measure of efficiency. The cohesive and shrinkage properties of the fine material are of utmost importance. Doctor Strahan reports kaolin as being an exceptionally good binder; and according to its constants kaolin possesses properties theoretically required by good binders. These constants are:

Liquid limit	60
Plasticity index	26
Shrinkage limit	36
Shrinkage ratio	1. 3
Centrifuge moisture equivalent	49
Field moisture equivalent	36

The liquid limit of 60, for instance, combined with a plasticity index of 26 indicates cohesive properties approaching those of an inert clay (curve 3). The shrinkage limit of 36 equals that of elastic silts and muck (curve 6). The centrifuge moisture equivalent of 49 indicates a water capacity slightly greater than that of average soils (curve 9). The field moisture equivalent of 36 indicates a resistance to water penetration characteristic of colloidal soils (curve 11): and the shrinkage limit being equal to the field moisture equivalent, the lineal shrinkage, like that of sand, is equal to 0.

It is clear that kaolin has both cohesion and water-retentive properties in moderate amount, relatively high resistance to water penetration when at the shrinkage limit, and negligible shrinkage properties. The constants possessed by kaolin may, therefore, serve as a basis for identifying good binders. These characteristics, it will be noted, indicate a plastic variety of the Group A–5 subgrade with an exceptionally low field moisture equivalent.

SUBGRADE SOILS MAY BE IMPROVED BY ADMIXTURES

It is natural that efforts should be made to increase the stability of certain varieties of subgrade soil by admixtures of suitable materials. The success of such efforts depends upon the manner and the extent to which unstable mixed materials differ in character from those which are stable. The changes which admixtures produce in the test constants indicate their effect on the physical properties of the soil.

If, according to mechanical analysis, the soil is deficient in coarse material and has appreciable plasticity, admixtures of coarse, granular materials, such as sand, slag, gravel, or crushed stone, may prove beneficial. It has been observed that the efficiency of rounded gravel may be increased considerably by either crushing or adding angular fragments to the rounded material.

If the active portion of the soil mortar, because of domination of clay, is high in both plasticity and shrinkage properties, admixtures of either porous silt or materials such as hydrated lime, diatoms, etc., having high shrinkage limits may serve to reduce the shrinkage properties without reducing too much the plasticity of the soil. Penetrative bituminous materials applied to reduce the moisture capacity of the clay may prove beneficial.

TABLE 8.—*Test results on several soils combined with various admixtures*

Soil No.	Admixture	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Volumetric change
				Limit	Ratio	Centrifuge	Field	
		P. ct.	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*
3,770	None	24	0	20	1.7	15	24	
	60 per cent A sand [1]	25	0			10	25	
	20 per cent mica	44	0	38	1.2	23	44	
	15 per cent peat	37	0	28	1.3	26	34	
	15 per cent diatoms	73	38	36	1.3	38	42	
	6 per cent colloids	39	20	16	1.6	32	28	
4,056	None	66	40	11	2.1	[2] 53	29	
	60 per cent A sand	36	20	17	1.9	23	27	
	20 per cent mica	70	42	16	1.8	63	51	
	15 per cent peat	72	38	15	1.8	66	47	
	15 per cent diatoms	89	54	28	1.6	75	52	
	6 per cent colloids	89	62	10	2.1	[2] 133	40	
5,041	None	65	36	14	1.9	55	50	68.4
	15 per cent hydrated lime	84	58	24	1.6	59	42	28.8

[1] Angular sand consisting of crushed diabase.
[2] Waterlogged.

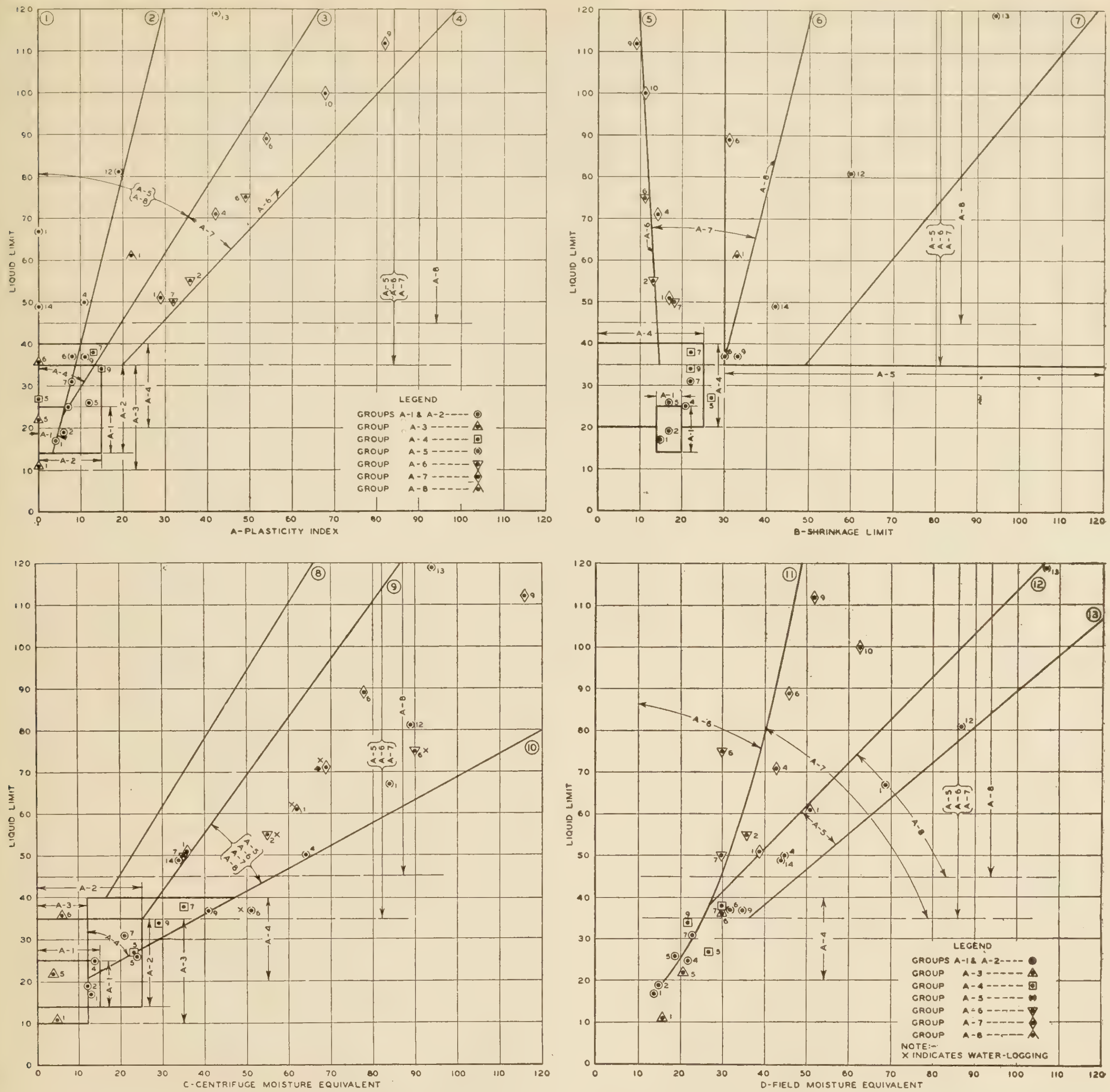


FIGURE 56.—THE SOIL IDENTIFICATION CHART

If the soil contains an abundance of coarse material and lacks clay and silt, cohesive materials obviously must be added. In this case either a proper clay binder may be added or the cohesionless coarse material may be treated with a highly penetrative bituminous material and covered with a light application of granular material.

If the soil, according to mechanical analysis, has proper grading but is low in both plasticity and shrinkage, and is to be placed on a very dry subgrade, the addition of a cohesive material may prove beneficial. Thus the gluey colloids which are detrimental to soil in large amounts may prove beneficial when present in very small amounts. Bentonite, for instance, added in the laboratory to a fine sandy loam in amounts not exceeding 3 per cent has the effect of introducing plasticity and resistance to erosion without increasing the shrinkage in detrimental amounts. Admixtures of bituminous materials, referred to above, may also serve this purpose very efficiently.

The data contained in Table 8 illustrate how the test constants disclose the influence exerted by different kinds of admixtures upon the characteristics of soils.

It is interesting to note that when mica and diatoms are added to the nonplastic soil, No. 3,770, both the shrinkage limits and the field moisture equivalents are very appreciably increased. The same is true when diatoms are added to the plastic soil, No. 4,056. When mica is added to the plastic soil only the field moisture equivalent is very appreciably increased.

It is interesting also to note that the volumetric change of soil No. 5,041 is 68.4 and that of the mixture of this soil and hydrated lime is only 28.8.

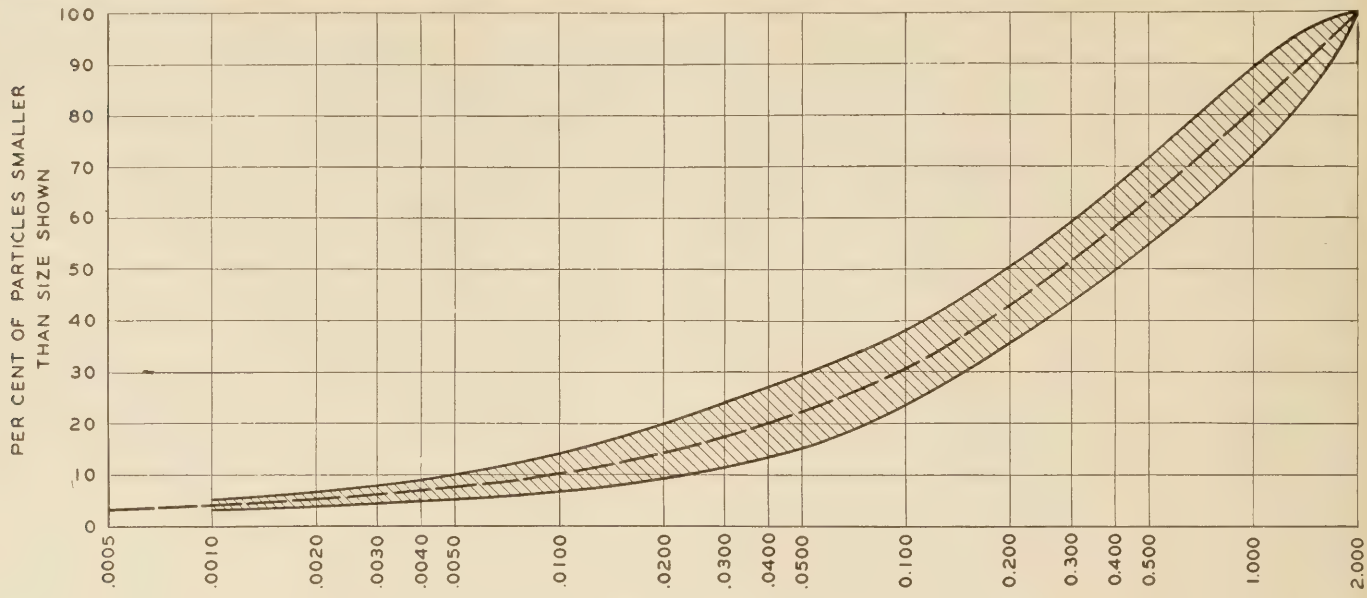


FIGURE 57.—GRADING OF GOOD SOIL MORTARS

GRADING AND CONSTANTS OF SUBGRADE GROUPS REVIEWED

The gradings of the various subgrade groups and the values of the test constants characteristic of them are repeated here as an aid to the discussion of soil identification, with which this part of the report is chiefly concerned.

Group A–1.—Grading: Material retained on the No. 10 sieve not more than about 50 per cent. The soil mortar, that fraction passing the No. 10 sieve, to consist of clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand, 45 to 60 per cent. Average effective size approximately 0.01 millimeters and uniformity coefficient greater than 15. The band shown in Figure 57 illustrates graphically the grading of good soil mortars.

Constants: Liquid limit not less than 14 nor greater than 25; plasticity index approximately equal to that indicated by curve 2 (fig. 56, A) and seldom greater than 8; shrinkage limit seldom less than 14 or greater than 20; and centrifuge moisture equivalent not apt to be greater than 15.

Fraction passing the No. 200 sieve—see constants of kaolin, p. 134, and Group A–5 subgrade below.

Group A–2.—Grading: Not less than about 55 per cent of sand in the soil mortar.

Constants: Liquid limit generally not less than 14 or greater than 35; a plasticity index of zero with a significant shrinkage limit or a plasticity index greater than zero and less than 15 with or without a significant shrinkage limit; centrifuge moisture equivalent not greater than 25.

Group A–3.—Grading: Effective size not likely to be less than 0.10 millimeters.

Constants: Liquid limit not appreciably greater than 35; no plasticity index; no significant shrinkage limit; centrifuge moisture equivalent less than 12.

Ability of sands to resist sliding when wet indicated as follows: Liquid limits of 10 to 14 signify beach and other rounded sands which slide easily; liquid limits of 30 to 35 indicate rough angular particles which do not slide easily. In addition, liquid limits when lower than field moisture equivalents indicate materials which flow under partial saturation; when equal to the field moisture equivalents, the liquid limits indicate average sands which flow under full hydrostatic uplift. Liquid limits greater than field moisture equivalents indicate rough-grained sands which flow only when in a state less consolidated than that represented by the field moisture equivalent. (See fig. 58.)

Group A–4.—Grading: Less than 55 per cent sand.

Constants: Liquid limit seldom less than 20 or greater than 40; plasticity index not greater than those indicated by curve 3; shrinkage limit not likely to be greater than 25; centrifuge moisture equivalent approaching those indicated by curve 10, between 12 and 50; when greater than liquid limit indicates varieties of soils inclined to be especially unstable in the presence of water; field moisture equivalent equal to or somewhat greater than those indicated by curve 11, with a maximum of about 30.

Increase in expansive properties generally indicated when shrinkage limits exceed 20 and approach those represented by curve 6; especially likely when field moisture equivalent exceeds centrifuge moisture equivalent.

Group A–5.—Grading: Less than 55 per cent sand. (Exceptions occur.)

Constants: Liquid limit usually greater than 35; plasticity index seldom greater than those indicated by curve 3; centrifuge moisture equivalent greater than 12, often lying between curves 9 and 10; not likely to water-log. (Exceptions occur.)

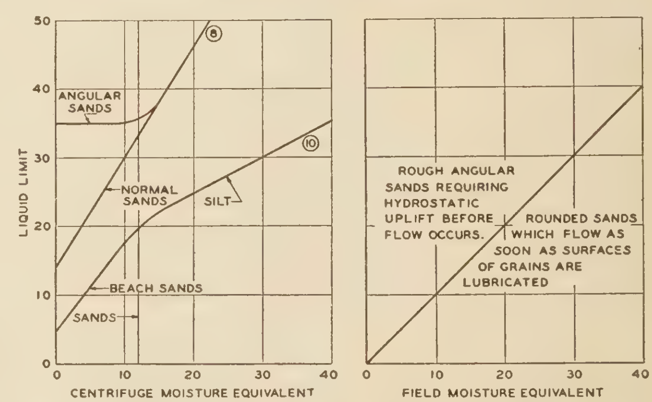


FIGURE 58.—SIGNIFICANT RELATIONS BETWEEN TEST CONSTANTS FOR GROUP A–3 SUBGRADE SOILS

Shrinkage limit generally greater than 30 and greater than 50 for very undesirable members of this group. May approach values indicated by curve 6 for silts containing peat and approach those indicated by curve 7 for soils containing either diatoms or mica in appreciable amount. Field moisture equivalent approaching those indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic soils containing mica or diatoms in appreciable amount. The kaolins, representing good binders, are members of group possessing relatively high plasticity indices and low field moisture equivalents.

Group A–6.—Grading: Seldom contains less than 30 per cent clay.

Constants: Liquid limit usually greater than 35; plasticity index approximately represented by curve 4; shrinkage limit not likely to be appreciably greatr than that indicated by curve 5; centrifuge moisture equivalent test generally productive of water-logging; likely to lie between curves 9 and 10; field moisture equivalent seldom exceeding those indicated by curve 11, but may be appreciably less for certain colloidal soils. Volumetric change generally greater than 17.

Group A–7.—Grading: Seldom contains less than 30 per cent clay.

Constants: Liquid limit usually greater than 35; plasticity index varies between those indicated by curves 3 and 4; shrinkage limit generally varies between those indicated by curves 5 and 6; centrifuge moisture equivalent varies between those indicated by curves 9 and 10; water-logging in centrifuge test may not occur even at very high moisture equivalents. Field moisture equivalent greater than those indicated by curve 11. Relatively low shrinkage limits with high field moisture equivalents indicate presence of colloidal organic matter. Relatively high shrinkage limits indicate the possibility of frost heave.

Group A–8.—Grading: Not significant.

Constants: Liquid limit greater than 45; plasticity index generally less than those indicated by curve 3; shrinkage limit indicated approximately by curve 6; centrifuge moisture equivalent between curves 9 and 10; field moisture equivalent likely to be greater than those indicated by curve 12.

Water-logging in the centrifuge test is characteristic of the mucks containing clay and colloids, whereas very high equivalents without water-logging are characteristic of peat not more than slightly decomposed.

NOMENCLATURE AND FORMULAS LISTED

The equations of the relationship curves 1 to 13, inclusive, and the basic formulas which have been developed in this report are listed below. The nomenclature for these equations is as follows:

$P.\ I.$ = plasticity index.
$L.\ L.$ = liquid limit.
$S.$ = shrinkage limit.
$C.\ M.\ E.$ = centrifuge moisture equivalent.
$F.\ M.\ E.$ = field moisture equivalent.
e = voids ratio.
e_o = voids ratio of dry sample.
V_v = volume of voids.
V_s = volume of soil particles.
V = volume of wet soil sample.
V_o = volume of dry soil sample.
C_o = volume change, percentage of V_o.
C_f = volumetric change, from $F.\ M.\ E.$, percentage of V_o.
$L.\ S.$ = lineal shrinkage, percentage of wet length.
M_w = weight of moisture.
W = weight of wet sample.
W_o = weight of dry sample.
w = moisture content, percentage of W_o.
w_v = moisture content, percentage of V_s.
G = specific gravity of soil particles.
P = porosity.
h = height of capillary rise in centimeters.
r = radius of capillary tube in centimeters.
a = width of pores in centimeters.
$S.\ F.$ = shrinkage force in grams per square centimeter.

EQUATIONS OF CURVES FOR SOIL IDENTIFICATION CHART

Curve 1.—$P.\ I. = 0$

Curve 2.—$P.\ I. = 0.25\ L.\ L.$

Curve 3.—$P.\ I. = \dfrac{L.\ L. - 14}{1.60}$

Curve 4.—$P.\ I. = \dfrac{L.\ L. - 14}{1.07}$

Curve 5.—$S = 21 - 1.1\sqrt{L.\ L. - \dfrac{L.\ L.^2}{800}}$

Curve 6.—$S = \dfrac{L.\ L. + 86}{4.1}$

Curve 7.—$S = \dfrac{L.\ L. + 26}{1.24}$

Curve 8.—$C.\ M.\ E. = \dfrac{L.\ L. - 14}{1.61}$

Curve 9.—$C.\ M.\ E. = 0.72\ L.\ L.$

Curve 10.—$C.\ M.\ E. = \dfrac{L.\ L. - 14}{0.55}$

Curve 11.—$F.\ M.\ E. = \sqrt{15.2\,(L.\ L. - 16.3)} + 9$

Curve 12.—$F.\ M.\ E. = \dfrac{L.\ L. - 10}{1.03}$

Curve 13.—$F.\ M.\ E. = \dfrac{L.\ L. - 4}{0.85}$

BASIC SOIL FORMULAS

$$e = \frac{V_v}{V_s} \quad \text{(1)}$$

$$= \frac{wG}{100} \quad \text{(10)}$$

$$P = \frac{e}{1+e} \times 100 \quad \text{(11)}$$

$$C_o = \frac{V - V_o}{V_o} \times 100 \quad \text{(2)}$$

$$= \frac{e - e_o}{1 + e_o} \times 100$$

$$C_f = (F.\ M.\ E. - S)\ R \quad \text{(18)}$$

$$= \frac{F.\ M.\ E. - S}{\dfrac{1}{G} + \dfrac{S}{100}} \quad \text{(19)}$$

$$L.\ S. = \frac{1 - \sqrt[3]{\dfrac{100}{C_f + 100}}}{0.01} \quad \text{(20)}$$

$$M_w = W - W_o \quad \text{(5)}$$

$$w = \frac{M_w}{W_o} \times 100 \quad \text{(4)}$$

$$w = \frac{W - W_o}{W_o} \times 100 \quad \text{(6)}$$

$$V_s = \frac{W_o}{G} \quad \text{(7)}$$

$$w_v = wG \quad \text{(8)}$$

$$h = \frac{0.153}{r} \quad \text{(See reference (1).)}$$

$$S.\ F. = \frac{0.306}{a} \quad \text{(14)}$$

$$S = w - \frac{V - V_o}{W_o} \times 100 \quad \text{(15)}$$

$$= \frac{100\ G - 100\ R}{RG}$$

$$R = \frac{W_o}{V_o} \quad \text{(16)}$$

$$= \frac{100\ G}{100 + SG}$$

SOIL SAMPLES ANALYZED BY GROUPS

In the pages which follow the test results from a large number of soil samples are tabulated and analyzed. With the aid of the soil identification chart, the place of each soil in the uniform subgrade classification is determined. From this procedure the reader may gain an idea of how identifications of this sort are accomplished in practice.

The plastic limit, shrinkage limit, centrifuge moisture equivalent, and field, moisture equivalent of a number of samples from each subgrade group are plotted as functions of the liquid limit on the soil identification chart, Figure 56, in order that the comparative relations of the constants may be studied in reference to the numbered curves 1 to 13.

Group A–1 and Group A–2 subgrades.—The constants and grading of samples of soil containing both coarse andfine materials are included in Tables 9 and 10, respectively.

Sample 1, which represents a stable soil from South Carolina, satisfies both the grading and the constant requirements of the Group A–1 subgrade.

TABLE 9.—*Group A–1 and Group A–2 subgrades, coarse fractions and binders. Constants of material passing the No. 40 sieve except as noted*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	17	4	15	1.9	13	14
2	19	6	17	1.8	12	15
3	20	7	18	1.8	13	16
4	25	7	21	1.7	14	22
5	26	12	17	1.8	24	19
6	25	5	-------	-------	23	23
7	31	8	22	1.6	21	23
8	25	-------	-------	-------	19	27
9	28	13	22	1.7	16	21
10	30	15	21	1.7	20	22
9–B [1]	49	29	-------	-------	27	-------
10–B [1]	58	34	-------	-------	36	-------
9–C [2]	24	-------	-------	-------	5	25
10–C [2]	23	-------	-------	-------	2	27

[1] Fraction passing No. 200 sieve.
[2] Fraction passing No. 40 sieve and retained on No. 200 sieve.

TABLE 10.—*Mechanical analyses of Group A–1 and Group A-2 subgrades*

Sample No.	Gravel particles larger than 2 millimeters	Mortar				
		Coarse sand 2.0 to 0.25 millimeter	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Good soil mortar	---------	[1] 45–60	---------	10–20	5–10	3–5
1	10	51	25	16	8	6
2	1	37	33	18	12	5
3	1	21	58	6	15	12
4	3	26	45	16	13	9
5	66	31	29	18	22	15
6	0	1	87	5	7	5
7	8	36	30	26	8	3
8	0	45	29	16	10	4
9	50	37	39	3	21	18
10	48	53	19	5	23	20

[1] Total sand 70 to 85 per cent.

Sample 2, which was obtained from a soil performing satisfactorily as fill in Escambia Bay, Fla., satisfies the constants but not the grading requirement of a Group A–1 material.

Its low centrifuge moisture equivalent of 12 indicates that the slight excess of clay above that generally indicating A–1 mortars is not likely to prove detrimental; and since the coarse sand in which this soil is deficient functions primarily to furnish the hardness required in wearing surfaces, this soil would probably prove highly stable as a subgrade.

The same is true for both sample 3, which was taken from a soil serving as a road surface in South Carolina, and sample 4, which was taken from a soil moderately stable when used as an untreated road surface in Madison County, Va., and highly stable when covered with bituminous surface treatments. (See fig. 59.)

FIGURE 59.—SURFACE-TREATED ROAD OF GROUP A–2 MATERIAL IN MADISON COUNTY, VA.

Sample 5, obtained from gravel used as road surface in Oklahoma; sample 6, from soil found by W. H. Dumont, of the United States Bureau of Fisheries, to be very unstable when occurring as bottom in Deep Creek, Ga.; and sample 7, from soil productive of frost heave in New Hampshire, fail to satisfy either the grading or the subgrade constant requirements of the Group A–1 material.

Sample 8, representing the Florida limerocks, satisfies the grading but not the constant requirements of the Group A–1 subgrade.

The high clay content of sample 5 is disclosed by the correspondingly high plasticity index of 12 and the centrifuge moisture equivalent of 24. The high plasticity index might indicate a desirable property, but the relatively high centrifuge moisture equivalent suggests water retentive properties not productive of stability. This detrimental property is probably offset to some extent by the fact that the field moisture equivalent is appreciably less than the centrifuge moisture equivalent.

The high silt content of sample 7 has produced a relatively high centrifuge moisture equivalent without raising the plasticity index in corresponding amount, one of the characteristics of soils productive of frost heave.

Special attention is called to sample 6, which, when submerged in water, has the properties of quicksand. This fact is disclosed by the nonuniform grading, which shows that the sample consists of 87 per cent fine sand, and the relatively high water-retentive property. The centrifuge moisture equivalent of this sample, containing but 12 per cent of particles smaller than 0.05 millimeter in diameter is practically the same as that of sample 5, which contains 40 per cent of particles smaller than 0.05 millimeter in diameter, and, furthermore, equals the field moisture equivalent which, in this case, can be assumed to indicate the amount of water required to fill the pores of the soil completely.

Sample 8 represents the nonplastic variety of the shell rocks. Tests performed on small beams of this material disclosed that when thoroughly dry this variety of limerock has practically no cohesion. Therefore, the stability of this material when used as a road surface must be due to the cohesion furnished by capillary pressure. Figure 60 shows how limerock occurs in nature.

ALTERNATE METHOD OF INVESTIGATING GRADED MATERIALS DISCUSSED

A somewhat elaborate method suggested for investigating the properties of mixed materials consists of testing the mixture as such and also testing separately both the nonexpansive and expansive soils of which it is composed, and thus investigating soil mixtures in a manner similar to that employed when investigating bituminous mixes, concrete, or other materials consisting of a binder and an aggregate.

A procedure of this kind includes (*a*) the determination of the relative resistance furnished by the soil mortar to water absorption by means of slaking tests; (*b*) the determination of the relative strength of the mortar when dried, by means of a crushing or impact test; (*c*) the determination of the grading of the soil mortar by means of the combined sieve and hydrometer method; (*d*) the determination of both the plastic and shrinkage properties of that fraction of the material passing the No. 200 (0.074 millimeter) sieve by means of the test constants; and (*e*) the determination of the character of that fraction of the sample retained on the No. 200 sieve according to the procedure employed in identifying the characteristics of Group A-3 subgrades discussed subsequently.

Samples 9 and 10, Tables 9 and 10, representing, respectively, a stable and an unstable soil in South Carolina, serve to illustrate how coarse and fine materials may be investigated separately.

Table 10, for instance, discloses that sample 9, compared with the average good soil, contains an excess of clay and fine sand and is deficient in silt and coarse sand. Sample 10, compared with sample 9, contains a greater amount of coarse and a less amount of fine sand.

The binders of the two samples, 9-B and 10-B, Table 9, are similar in character, although the constants of 10-B are higher than those of 9-B. Both, however, have plasticity indices greater than that of kaolin (greater than that indicated by curve 3), combined with relatively low water retentive properties (curve 9). This indicates open-structure clays unlikely to shrink or expand in appreciable amount.

The constants of the sand fractions 9-C and 10-C, Table 9, are not radically different. Sample 9-C, however, which contains the greater amount of fine sand, has the higher centrifuge moisture equivalent. The liquid limit of sample 9-C is practically equal to the amount of water (field moisture equivalent) required to saturate the sample completely, whereas the liquid limit of sample 10-C is reached at a moisture content slightly below that required to saturate the soil completely.

From these evidences it would seem that the excellent quality of the binder primarily accounts for the fact that the soil represented by sample 9 can contain so large a clay content and yet remain stable.

That the same high clay content proved detrimental to the soil represented by sample 10 may be explained by the fact that, because of difference in grading, the surface area of the grains in the sand sample 9 may exceed by more than 40 per cent that of the grains in the sand fraction sample 10. Therefore, if the colloidal material, approximately the same in both samples, can be considered as a glue which coats the surfaces of the sand grains, sample 9, with the larger surface area, will cause the glue to be distributed in films thinner than those in sample 10. For this reason the quantity of glue (colloidal material) which causes the films to be excessively thick in sample 10 may not prove detrimental to sample 9.

Group A-3 subgrades.—The samples included in Tables 11 and 12 satisfy in both grading and constants the requirements of the cohesionless sand subgrades which provide good drainage and which do not have the tendency to heave under frost action.

Figure 60.—Examples of Shell Rock in Natural Location: A, Chalky Limerock of the Ocala, Fla., Region. B, Cochina Rock Near Ormond Beach, Fla. C, Ogus Rock, Lower East Coast of Florida

TABLE 11.—*Group A–3 subgrades; Constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	11	0			5	16
2	20	0			4	25
3	19	0			3	17
4	20	0			3	21
5	22	0			4	21
6	36	0			6	30

TABLE 12.—*Mechanical analyses of Group A–3 subgrades*

Sample No.	Gravel particles larger than 2.0 milimeters	Coarse sand 2.0 to 0.25 milimeter	Fine sand 0.25 to 0.05 milimeter	Silt 0.05 to 0.005 milimeter	Clay below 0.005 milimiter	Colloids below 0.001 milimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	74	18	3	5	3
3	0	25	72	1	2	1
4	0	20	78	1	1	0
5	0	41	56	2	1	0

FIGURE 61.—THREE-INCH GROUTED BRICK ROAD CONSTRUCTED ABOUT 1916 ON GROUP A–3 SUBGRADE IN FLORIDA. STILL IN SERVICE. CONCRETE SHOULDERS CONSTRUCTED IN 1921

Sample 1 was obtained from a Florida sand which serves excellently as subgrade, when prevented from flowing laterally, for relatively thin road surfaces. (See figs. 61 and 62.) Sample 2 represents that fraction of Potomac River sand passing the No. 20 and retained on the No. 100 sieve. Sample 3 was obtained from a California sand which serves excellently as a subgrade for concrete pavements 4½ inches thick (*23*). Sample 4 was taken from a Minnesota sand which becomes highly stable when treated with bituminous materials possessing penetrative properties in high degree, and covered with a thin application of granular material. Sample 5 is a New Hampshire sand which furnishes excellent support when treated in a manner similar to that described for sample 4. Sample 6 represents that fraction of crushed diabase passing the No. 20 and retained on the No. 100 sieve.

According to the significance of the relation existing between the liquid limits and the centrifuge moisture equivalents of these soils, the soils represented by sample 1 would be expected to have very low stability and those represented by sample 6 would be expected to have very high stability in the presence of water.

Additional information on the character thees of soils is furnished by the relation between the liquid limit and the field moisture equivalent. Assuming that the field moisture equivalents of sand equal the amounts of water required to fill the pores of the sands completely when compressed by a very small but constant pressure, the glacial sands represented by samples 4 and 5, Table 11, are likely to flow when completely saturated (full hydrostatic uplift). The beach and river sands of samples 1 and 2 will flow when the grains are lubricated, rather than as a result of full hydrostatic uplift. The angular fragments of sample 6 require water in amounts greater than the field moisture equivalent to cause flow.

FIGURE 62.—SURFACE TREATED LIMEROCK ROAD SIX INCHES THICK IN FLORIDA

The fact that the liquid limit is greater than the field moisture equivalent indicates that, in order to flow, the angular particles must exist in a state looser than that represented by the field moisture equivalent. It appears from these facts that sample 1, when in the presence of water, is likely to be the least, and sample 6 the most stable of the sands listed in Tables 11 and 12.

Group A–4, subgrades.—Samples 1 to 9, inclusive, Tables 13 and 14, have constants indicative of those soils which have the tendency to heave under frost action and to lose stability as a result of water absorption even when not manipulated.

Samples 1 and 2 were made up, respectively, from commercial rotten stone and chalk. Although these materials are not natural soils they have the properties common to the A–4 subgrades. Sample 3 was composed of marl from Florida. Marl when kept dry and not subjected to frost action serves excellently as base course material. Samples 4 and 5 were obtained from New Hampshire silts which have been observed to heave in detrimental amounts under frost action. Sample 6 was obtained from a silt found in Missouri. Pavements laid on this soil have been observed to crack in appreciable amount. Samples 7 and 8 represent soils in the Minnesota frost-boil area which heave under frost and lose stability during the spring thaws. (See fig. 63.) Sample 9 was composed of lithopone, which has been found by Prof. Stephen Taber to suffer important frost heave.

In grading this soil should belong to the highly colloidal clays of either Group A–6 or A–7. The plasticity index of 15 compared with the liquid limit of 34 is slightly higher than those designating the A–4 subgrades. The relatively high shrinkage limit and low field moisture equivalent, indicating that this material is unlikely to shrink in appreciable amount, suggests the performance of silt instead of clay. Com-

TABLE 13.—*Group A–4 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	32	11	22	1.7	[1] 39	24
2	29	6	28	1.5	34	26
3	38	12	24	1.6	37	30
4	25	0	25	1.6	19	24
5	27	0	27	1.6	23	27
6	32	10	24	1.7	29	28
7	38	13	22	1.6	35	30
8	32	14	22	1.7	33	27
9	34	15	22	2.0	29	22

[1] Water-logged.

TABLE 14.—*Mechanical analyses of Group A–4 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	7	57	36	17
2	0	0	13	37	50	3
4	1	14	34	28	24	1
5	0	13	21	42	24	2
6	0	5	10	58	27	12
7	1	7	20	52	21	4
8	1	9	26	46	19	4
9	0	0	2	3	95	67

parison of the constants of this material with those of statistical soils also discloses its inactivity.

A statistical soil, for instance, containing 95 per cent clay has a liquid limit of 132, a plasticity index of 74, a shrinkage limit of 10, a centrifuge moisture equivalent of 95, and a field moisture equivalent of 51. Thus, lithopone, irrespective of its grading, has relatively very low cohesive properties, and this fact, combined with its negligible shrinkage properties and a liquid limit smaller than 40, causes lithopone to be grouped with the silts.

Group A–5 subgrades.—The constants and the grading characteristic of Group A–5 material are shown in Tables 15 and 16. In grading, these samples contain either clay or silt in dominating amounts. Only one sample has a liquid limit smaller than 35; all samples have plasticity indices smaller than those indicated by curve 3 and all but two samples have shrinkage limits equal to or greater than 30. The four samples whose shrinkage limits are not given have values of this quantity higher than the liquid limit.

Samples 1, 2, and 3 were made, respectively, from commercial hydrated lime, pumice, and talc. Samples 4 and 5 were made from barium sulphate and reground quartz, respectively. Professor Taber found the former to heave very appreciably and the latter to heave but very little as a result of frost, in experiments performed in the laboratory. Samples 6 and 7 were composed of silt found in St. Peter, Minn., which, according to F. C. Lang, heaved several feet during the winter of 1928-29. Samples 8 and 9 were obtained from New Hampshire silts which have been found by W. F. Purrington to heave in appreciable amount under frost action. Samples 10 and 11 were made up from Oregon silts which have been observed by R. H. Baldock to suffer important frost heave.

In grading, samples 6, 7, and 8, which are comparable to barium sulphate (sample 4), contain clay in dominating amount. Samples 9, 10, and 11, which are comparable to reground quartz (sample 5), contain silt in dominating amounts. Sample 9 contains mica, while samples 10 and 11 contain an appreciable amount of organic matter.

The soils represented by samples 6 to 11, inclusive, in addition to appreciable frost heaving, are likely to lose stability during thaws. In addition, these soils are capable of raising water in detrimental amounts through great heights during frost action.

Samples 12 and 13 were obtained from Maryland soils containing diatoms in appreciable amount. Soils of this character, especially when their shrinkage limits are equal to or greater than 50, are almost sure to produce pavement failure because of their high porosity.

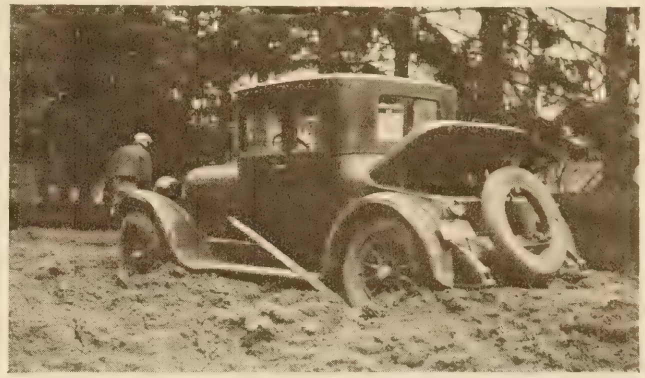


FIGURE 63.—TYPICAL FROST BOIL CAUSED BY LACK OF STABILITY IN SOIL

TABLE 15.—*Group A–5 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	67	0	----------	----------	84	69
2	58	0	----------	----------	36	62
3	43	17	41	1.3	48	36
4	50	11	----------	----------	[1] 64	45
5	47	0	----------	----------	22	28
6	37	8	30	1.5	[2] 51	32
7	36	9	28	1.5	[2] 40	31
8	45	18	37	1.4	54	37
9	37	11	33	1.5	41	35
10	40	8	33	1.3	36	37
11	43	9	37	1.3	38	41
12	81	19	60	0.9	89	87
13	119	42	94	0.7	94	107
14	49	0	42	1.2	34	44
15	34	12	26	1.5	25	30

[1] Tendency to waterlog. [2] Waterlogged.

TABLE 16.—*Mechanical analyses of Group A–5 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeter	Coarse sand 2.0 to 0.25 millimeter	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	1	7	86	([1])	([1])
2	0	0	10	78	12	8
3	0	0	9	74	17	7
4	0	0	1	6	93	26
5	0	0	8	78	14	4
6	0	2	5	16	77	17
7	0	0	14	27	59	40
8	0	1	6	32	61	8
9	0	1	5	60	24	2
10	3	6	23	57	14	0
11	3	6	27	60	7	0
12	0	0	29	42	29	12
13	0	0	40	28	32	9

[1] Flocculated.

With a specific gravity of 2.0, for instance, and a shrinkage limit equal to 50, these soils in their dried state have voids in amount equal to the volume occupied by the soil particles. Figure 64 shows a dried pat of sample 13 floating on the surface of water.

Samples 14 and 15 were taken from micaceous soils in Maryland. These samples are representative of the elastic subgrades which are productive of the early cracking in concrete pavements illustrated in Figure 15, A, Part I, of this report.

FIGURE 64.—DRIED PAT OF DIATOMACEOUS EARTH (SAMPLE 13, TABLE 15) FLOATING IN WATER AFTER BEING COATED WITH SHELLAC. APPARENT SPECIFIC GRAVITY LESS THAN ONE

A comparison of Tables 14 and 16 shows that it would be impossible to distinguish Group A–4 from Group A–5 subgrades on the basis of the mechanical analyses alone. The difference in the constants (Tables 13 and 15), however, indicates a difference in the characteristics of the soils. The higher liquid limits combined with the high shrinkage limits and field moisture equivalents differentiate those soils possessing elasticity (Group A–5) from those which are compressible (Group A–4).

Samples 4 and 7, according to the mechanical analyses, contain colloids in amount sufficient to produce high plasticity and shrinkage properties in appreciable amount, were the colloids active. These soils, however, according to their constants shown in Table 15, have low plasticity and negligible shrinkage properties and, consequently, should be grouped with the A–5 instead of the A–7 subgrades.

It is interesting to note that two soils (samples 7 and 9) which are likely to heave in appreciable amount have similar constants in spite of the fact that sample 7 contains 40 per cent of particles of colloidal size and 59 per cent of clay, whereas sample 9 contains but 2 per cent of colloidal particles and 24 per cent of clay. This is further evidence that grain size alone does not control the performance of subgrade soils.

Group A–6 subgrades.—The samples included in Tables 17 and 18 illustrate the constants and the grading characteristic of the nonelastic colloidal clay subgrades. Sample 1 was obtained from a colloidal clay soil which has proved troublesome because of sliding in a cut in Missouri, and sample 2 from a colloidal clay soil which has caused similar difficulties when used for fill material, in the same State. Samples 3 and 4 were composed of colloidal clays furnished by a survey of the soil existing under the Potomac River at Washington, D. C. These soils were considered unfit for use as hydraulic fill material. Sample 5 represents a very highly colloidal clay soil productive of landslides in Virginia. This soil contains about 30 per cent of material so fine that it remains in suspension for weeks. If located on an impervious undersoil, this soil when in a soft condition acts as a lubricant facilitating the sliding of the upper soil layers. (See Fig. 65.) Sample 6 was obtained from a colloidal soil from Texas, and sample 7, when occurring as subgrade, produced failure in a limerock base course being constructed on a road north of Gainesville, Fla.

TABLE 17.—*Group A–6 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	51	31	13	1.9	[1] 56	34
2	55	36	13	1.9	[1] 55	36
3	58	40	12	1.9	[1] 45	28
4	57	40	12	1.9	[1] 45	28
5	132	101	11	1.9	[1] 178	33
6	75	49	11	2.0	[1] 90	30
7	50	32	18	1.7	35	30

[1] Waterlogged.

TABLE 18.—*Mechanical analyses of Group A–6 subgrades*

Sample No.	Gravel-particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeters	Silt 0.05 to 0.005 millimeters	Clay below 0.005 millimeters	Colloids below 0.001 millimeters
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	1	52	40	15
2	0	0	11	57	32	16
3	0	19	30	11	50	24
4	16	25	12	30	43	27
5	0	3	3	6	88	78
6	0	3	20	29	48	24
7	0	28	32	17	23	10

The relatively large plasticity indices and the relatively small shrinkage limits and field moisture equivalents, combined with waterlogging in the centrifuge test, identify samples 1 to 6, inclusive, as containing clay highly active in character. This is emphasized by the mechanical analyses, which disclose that only one of the six samples contain more than 50 per cent of clay.

Sample 7, on the basis of its grading and its shrinkage limit would be classed an A–2 subgrade. Its volumetric change of 20.4 $((30-18)\times 1.7)$ and its plasticity index of 32 for a combined clay and silt content not larger than 40 definitely place this soil with the colloidal clays. The clay contained in this sample was similar in stickiness to chewing gum. This clay is the only subgrade material found thus far in the subgrade investigations which has an activity practically equal to that of bentonite. It is interesting to note that the plasticity index of this sample, which contains but 23 per cent of clay, is more than double that of lithopone, which contains 95 per cent of clay.

The presence of gluey colloids generally serves to protect the Group A–6 subgrades from detrimental frost heaving. These soils are likely to contain relatively large amounts of the unfreezable water productive of frost heave. The rate at which capillary moisture moves through them, however, is so slow that detrimental heave will probably not occur unless well-defined seepage planes furnish the necessary moisture at the required rate.

Group A–7 subgrades.—The samples contained in Tables 19 and 20 illustrate the grading and the constants characteristic of those clay subgrades which are likely to be elastic under certain prevalent conditions. Sample 1, furnished by F. V. Reagel, was taken from an expansive clay on which a concrete pavement in Missouri cracked in appreciable amount during the setting period of the concrete. Sample 2, furnished by W. C. McKnown, was obtained from an expansive clay in Kansas on which similar cracking occurred in a concrete pavement. Sample 3, furnished by W. D. Ross, was obtained from an expansive clay in Colorado on which similar cracking occurred in the pavement.

TABLE 19.—*Group A–7 subgrades; constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	51	29	17	1.7	36	39
2	39	20	15	1.9	28	27
3	42	24	14	1.9	35	34
4	71	42	14	1.8	[1] 69	43
5	63	38	17	1.8	[1] 60	46
6	89	54	31	1.9	[2] 78	46
7	67	37	20	1.7	[2] 46	39
8	68	42	12	2.0	[1] 67	47
9	112	82	9	2.0	116	52
10	100	68	11	2.0	[1] 132	63
11	83	53	13	1.9	[1] 72	64

[1] Waterlogged. [2] Tendency to waterlog.

TABLE 20.—*Mechanical analyses of Group A–7 subgrades*

Sample No.	Gravel particles larger than 2.0 millimeters	Coarse sand 2.0 to 0.25 millimeters	Fine sand 0.25 to 0.05 millimeter	Silt 0.05 to 0.005 millimeter	Clay below 0.005 millimeter	Colloids below 0.001 millimeter
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	0	0	7	55	38	24
2	0	1	14	52	33	16
3	0	2	5	45	48	21
4	0	2	9	43	46	22
5	0	1	13	43	43	24
6	0	0	2	6	92	78
7	0	0	5	23	72	29
8	0	3	8	35	54	27
9	0	0	4	15	81	18
10	0	1	9	54	36	16
11	0	1	14	28	57	21

These three soils, it will be noted, are similar in several respects. They all contain approximately 50 per cent of silt; the plasticity indices of the three soils bear similar relationships to their liquid limits, being slightly less than those represented by curve 4; the centrifuge moisture equivalents of all three soils are relatively small, being approximately equal to those represented by curve 9; and the field moisture equivalents are either approximately equal to or greater than the centrifuge moisture equivalents. It will be recalled that the micaceous silts of the Group A–5 subgrade, on which cracking occurred during the early age of the concrete, also have field moisture equivalents generally larger than the centrifuge moisture equivalents.

Samples 4 and 5 were composed of gumbos from the Red River Valley, Minn., which do not suffer detrimental frost heave and which are stabilized when oil-treated and covered with granular material.

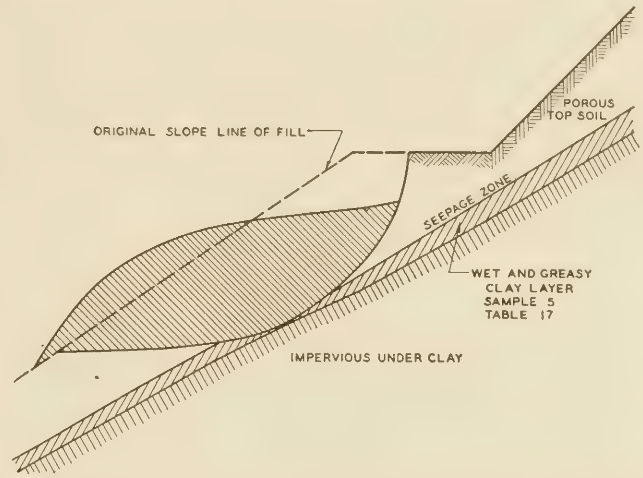


FIGURE 65.—DIAGRAM ILLUSTRATING LANDSLIDE CAUSED BY PRESENCE OF HIGHLY COLLOIDAL CLAY SUCH AS SAMPLE 5, TABLE 17

Sample 6 was composed of kadox, which, according to laboratory experiments performed by Professor Taber (*24*), is productive of important frost heave. Except for low colloidal activity and high shrinkage limit this material seems to be a normal A–7 subgrade. The shrinkage limit of 31, however, is approximately equal to those of the Group A–5 silts of New Hampshire (samples 8 and 9, Table 15) in which detrimental heave due to frost is likely to occur.

The relative activity of the colloids in the two samples, 5 and 6, is disclosed when their constants are expressed as decimals of those characteristic of the average or statistical soils. Thus the plasticity index of sample 5 is 1.31 times that of an "average" soil containing 43 per cent of clay, and the plasticity index of sample 6 is only 0.76 times that of an "average" soil containing 92 per cent clay. Kadox, therefore, possesses both the relatively inactive colloids and the high porosity productive of frost heave.

Sample 7, obtained from a soil in Michigan similar to sample 6, has a relatively low plasticity index, only 0.69 times that of an "average" soil containing 72 per cent clay; and also a relatively high shrinkage limit, 1.82 times that possessed by the average soil referred to. This soil has been observed to have detrimental elasticity in dry weather. Increasing the moisture content of the soil, however, causes a decrease in its elasticity. This is true also for certain varieties of the micaceous silts.

This soil displays the properties indicative of frost heave. The current surveys, however, have not yet disclosed whether or not such heave occurs.

Sample 8 was obtained from a soil which became exceedingly troublesome because of sliding when used in a high fill in Missouri. The organic matter which causes this soil to be placed in the A–7 group has decomposed to such an extent that its presence is disclosed only by the relatively large field moisture equivalent.

Sample 9 represents a flocculated, highly colloidal soil from Mississippi. Only the absence of water-logging with a centrifuge moisture equivalent as large as 116 prevents this soil from being grouped with the A–6 subgrades. This soil, which contains both lime and gypsum, supports concrete slabs which have warped in detrimental amounts. Soil of this character can not be used efficiently in fills and in cuts it should be separated from concrete pavements by a good topsoil base course at least 2 feet thick.

Sample 10 was made up from a Texas black waxy soil. Its high field moisture equivalent causes it to be grouped with the A–7 subgrades. Sample 11 is another soil of the gumbo type which proved troublesome when used in fills in Arkansas.

FIGURE 66.—DRY LAND FILL CONSTRUCTED ON GROUP A–8 SUBGRADE IN VIRGINIA

Group A–8 subgrades.—Constants of representatives of the soft peats and mucks are contained in Table 21. The grading of these materials is not significant.

TABLE 21.—*Group A–8 subgrades; Constants of material passing the No. 40 sieve*

Sample No.	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent	
			Limit	Ratio	Centrifuge	Field
	Per cent	*Per cent*	*Per cent*		*Per cent*	*Per cent*
1	61	22	33	1.4	[1] 62	51
2	62	26	38	1.4	[1] 56	47
3	59	19	31	1.4	68	51
4	265	0	141	0.5	263	265
5	445	0	187	0.3	395	440

[1] Waterlogged.

Samples 1 and 2 were obtained from Potomac River bottom muck which failed to support hydraulic fills without displacing laterally. The high shrinkage limits and high field moisture equivalents indicate the presence of partly decomposed organic matter. The relatively high plasticity indices and the water logging in the centrifuge test disclose the presence of clay or colloidal organic matter which in combination with the partly decomposed organic matter comprises muck. Sample 3 was obtained from Minnesota muck, which has a tendency to displace laterally when supporting fills. The presence of sand in this material may account for the absence of water logging in the centrifuge test. Sample 4 was composed of a peat which failed to support a dry land fill in Virginia (fig. 66) and sample 5 from a peat which has proved inadequate to support fills in Minnesota.

Samples 4 and 5 illustrate the very high water-absorptive properties characteristic of the peat soils which have not yet reached the colloidal state by decomposition. That these two soils have not yet reached this state is indicated by the fact that their plasticity indices are equal to zero and by the absence of water logging in the centrifuge test.

CONCLUSIONS SUMMARIZED

The foregoing discussion serves to emphasize that relatively few and comparatively simple laboratory tests may serve to identify fairly accurately the important characteristics of subgrade soils. Consequently these tests may serve to identify the dominating constituents composing the soils and to suggest the proper corrective measures to be used in pavement construction.

The following generalizations, based on the data given in the preceding pages, illustrate the service performed by the test constants in identifying the characteristics of subgrade soils.

Graded materials having centrifuge moisture equivalents greater than 15 have been found to be related to loss of subgrade stability in the presence of water.

Groups A–4, A–5, and A–7 subgrades having field moisture equivalents approximately equal to or greater than the centrifuge moisture equivalents have been found to be conducive to the cracking which occurs in pavements during the early life of the concrete.

Groups A–2, A–4, and A–5 subgrades having relatively high centrifuge moisture equivalents and Group A–7 subgrades having exceptionally high shrinkage limits seem to favor detrimental frost heave.

Group A–3 subgrades and groups A–6 and A–7 subgrades having relatively high plasticity indices are unlikely to heave under frost action.

Groups A–6 and A–7 subgrades are likely to shrink and expand in appreciable amount.

Group A–7 subgrades, having relatively high shrinkage limits, and Group A–5 subgrades, because of their elasticity, are likely to prove troublesome in the preparation of the subgrade.

The plastic varieties of the Group A–5 subgrades, with exceptionally low field moisture equivalents, generally have the properties required in good binders for sand-clay and topsoil roads.

The principal purpose of this report is to describe a method according to which soil research yielding profitable results may be performed. It should serve also to illustrate (*a*) the effort which may be required to interpret the test constants properly, (*b*) the character of the information which may be obtained by the intelligent use of the test constants, (*c*) the necessity for understanding the full significance of each constant, and (*d*) the impossibility of stating in simple terms the general procedures by means of which the constants of all soils may be readily interpreted.

It is expected that future invetigations will improve the procedure for making soil tests, and will increase the precision and facility with which soils may be identified by the use of test constants.

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Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
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Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID AND EMERGENCY ROAD CONSTRUCTION

AS OF

JUNE 30, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION						BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated Total Cost	Federal-Aid Allotted	Emergency Advance Fund	Mileage			Estimated Total Cost	Federal-Aid Allotted	Emergency Advance Fund	Mileage				
					Initial	Stage*	Total				Initial	Stage*	Total		
Alabama	2,115.8	$ 7,537,014.55	$ 3,671,288.23	$ 1,564,430.71	236.7	114.8	351.5	$ 93,312.53	$ 46,656.26	$ 34,691.58	1.4	2.7	4.1	$ 3,291,396.79	Alabama
Arizona	841.9	6,419,231.92	4,295,941.88	1,081,000.52	260.0	229.3	489.3	84,143.72	54,813.77	13,504.20	2.4	5.1	7.5	92,674.36	Arizona
Arkansas	1,737.7	9,139,166.13	4,354,305.88	1,373,157.00	235.6	102.1	337.7	1,491,745.71	713,622.89		23.7	29.9	53.6	114,579.53	Arkansas
California	1,928.0	12,395,686.82	5,206,186.73	2,672,268.60	293.4	103.5	396.9	1,804,148.84	663,854.92	291,467.14	60.4		60.4	926,385.10	California
Colorado	1,343.3	5,781,305.19	3,115,215.63	1,316,888.20	206.2	89.0	295.2	328,975.27	184,752.46	51,871.71	15.9	21.4	37.3	1,968,342.74	Colorado
Connecticut	256.9	4,952,568.48	1,794,286.20	520,491.00	49.0		49.0	377,652.56	176,946.82		5.5		5.5	4,388.65	Connecticut
Delaware	306.0	1,041,247.28	519,869.13	400,000.00	52.6		52.6	180,024.00	90,012.00		7.4		7.4	73,516.71	Delaware
Florida	540.0	6,424,859.85	3,007,205.42	1,029,346.35	175.5		175.5	102,816.84	51,408.41	13,258.00	13.3		13.3	1,604,812.74	Florida
Georgia	2,795.0	9,004,537.69	4,302,305.86	1,818,777.17	303.1	113.8	416.9	1,555,393.35	730,085.16	48,300.00	77.7	18.2	95.9	1,049,771.28	Georgia
Idaho	1,281.9	4,085,734.91	2,297,771.02	869,534.92	271.2	32.9	304.1	987,824.36	574,435.01	137,770.21	58.3	55.9	114.2	395,029.73	Idaho
Illinois	2,267.7	25,622,268.77	11,785,892.09	2,376,700.00	769.3	41.1	810.4	8,257,424.68	3,778,078.65	1,012,300.00	271.9	4.7	276.6	279,005.33	Illinois
Indiana	1,579.0	9,209,958.89	4,536,328.72	1,030,313.38	281.3		281.3	3,030,777.18	1,453,338.00	353,212.70	129.5		129.5	755,222.13	Indiana
Iowa	3,153.1	7,386,346.86	3,144,075.10	1,993,369.00	167.3	72.1	239.4	549,700.61	182,458.80	123,000.00		17.4	17.4	4,059.10	Iowa
Kansas	3,148.8	8,883,373.59	4,247,356.40	1,535,965.37	505.6	55.8	561.4	1,804,249.75	868,140.63	175,947.54	113.0	46.2	159.2	386,026.70	Kansas
Kentucky	1,519.0	7,439,643.91	3,259,483.02	1,195,190.78	284.4	131.1	415.5	880,912.21	428,092.95	230,600.00	60.8	19.8	80.6	183,559.55	Kentucky
Louisiana	1,418.9	8,422,172.79	4,005,813.00	1,137,927.00	245.9	19.3	265.2	120,355.82	58,515.30	10,000.00	5.1	.6	5.7	253,076.15	Louisiana
Maine	603.2	4,337,397.40	1,642,395.30	637,799.00	101.0		101.0	1,084,671.79	470,583.82	43,000.00	19.9		19.9	612,261.07	Maine
Maryland	707.0	1,652,189.40	759,448.81	582,772.49	47.9	4.2	52.1	405,155.40	202,577.66	68,339.88	17.7		17.7	91,940.51	Maryland
Massachusetts	724.2	9,210,169.31	2,276,626.32	802,509.00	78.9		78.9	1,467,104.61	563,800.20	266,000.00	17.0		17.0	1,597,158.35	Massachusetts
Michigan	1,804.8	10,793,690.64	4,466,907.21	1,108,000.00	314.4	23.9	338.3	2,142,332.29	1,024,613.00	257,500.00	81.7	27.3	109.0	1,973,791.60	Michigan
Minnesota	3,987.3	7,643,473.34	3,237,288.25	1,954,993.00	74.3	284.5	358.8	1,278,434.70	546,474.80	295,000.00	5.6	49.6	55.2	27,432.47	Minnesota
Mississippi	1,772.7	4,089,403.10	2,013,896.18	1,401,409.53	182.0	80.5	262.5	402,117.91	201,066.84	30,908.89	2.5	18.3	20.8	3,759,645.69	Mississippi
Missouri	2,659.5	9,829,670.52	3,905,276.93	2,264,272.94	246.8	65.0	311.8	1,979,887.13	846,928.77	213,472.79	61.2	25.8	87.0	30,847.48	Missouri
Montana	1,835.1	11,719,226.48	6,597,231.21	1,637,674.28	888.2	157.9	1,046.1	438,573.99	243,222.65	7,500.00	36.9	29.2	66.1	2,120,118.98	Montana
Nebraska	3,848.9	9,110,895.80	4,268,484.24	1,577,666.73	245.9	199.7	445.6	1,653,248.55	769,179.23	108,165.89	54.9	68.9	123.8	904,033.63	Nebraska
Nevada	1,092.0	2,946,231.13	1,983,134.43	813,056.02	108.0	248.6	356.6	628,107.07	395,796.58	200,366.44	18.8	24.8	43.6	204,157.11	Nevada
New Hampshire	392.9	1,343,033.46	515,208.60	270,000.00	26.1	1.0	27.1	465,120.12	127,744.36	130,000.00	3.4	7.3	10.7	94,801.98	New Hampshire
New Jersey	555.2	6,209,545.44	1,794,968.33	1,103,770.59	74.7		74.7	144,716.24	3,420.00		.2		.2	1,177,568.27	New Jersey
New Mexico	1,927.9	6,820,755.52	4,117,269.64	1,259,480.49	251.5	145.5	397.0	107,277.63	67,700.35		.2		.2	86,785.38	New Mexico
New York	2,684.0	39,415,270.33	13,701,573.50	3,714,566.00	690.9	7.0	697.9	6,191,200.00	2,428,525.00	230,000.00	115.9		115.9	15,421.72	New York
North Carolina	1,970.6	6,565,115.87	3,126,179.57	1,626,473.82	213.1	35.4	248.5	1,023,035.04	503,233.00	233,527.78	29.0	36.4	65.4	1,608,127.62	North Carolina
North Dakota	4,363.2	4,377,946.70	2,233,117.38	961,858.98	453.3	457.4	910.7	877,431.89	438,715.84	118,900.00	90.4	194.9	285.3	1,087,249.83	North Dakota
Ohio	2,553.6	15,298,336.93	4,780,217.25	1,830,069.07	231.1	21.6	252.7	5,056,660.88	1,964,856.48	1,168,468.93	95.9	23.4	119.3	1,236,446.47	Ohio
Oklahoma	1,981.3	7,764,820.32	3,678,809.67	1,601,769.48	298.4	117.7	416.1	1,078,316.17	579,499.44	308,814.37	39.0	13.9	52.9	8,011.61	Oklahoma
Oregon	1,267.7	7,564,517.07	4,175,255.21	1,012,430.03	246.8	90.3	337.1	728,600.13	362,847.87	292,423.05	49.1	8.4	57.5	442,216.97	Oregon
Pennsylvania	2,664.4	12,715,867.36	5,194,853.56	2,713,936.94	251.7		251.7	2,700,355.73	1,295,663.77	787,079.41	96.4		96.4	1,151,378.69	Pennsylvania
Rhode Island	215.4	2,621,715.53	976,610.80	400,000.00	42.1		42.1	91,465.74	45,732.86		.1		.1	98,072.80	Rhode Island
South Carolina	1,853.5	5,423,297.93	2,457,109.74	1,114,636.00	78.2	152.8	231.0							242,641.12	South Carolina
South Dakota	3,741.6	6,206,502.83	3,334,679.87	1,220,359.54	419.4	188.4	607.8	512,582.94	254,606.69	37,000.00	15.1	47.1	62.2	383,082.29	South Dakota
Tennessee	1,461.4	4,542,803.15	2,237,133.62	1,741,881.99	193.7	17.4	211.1	380,641.70	190,320.84		26.0		26.0	1,838,998.09	Tennessee
Texas	6,979.5	20,038,372.56	9,178,189.78	4,649,762.26	858.4	324.4	1,182.8	2,777,023.12	1,226,795.58	435,636.91	87.6	51.2	138.8	3,256,872.24	Texas
Utah	1,041.5	2,204,486.97	1,284,752.41	608,169.04	133.9	38.9	172.8	819,142.92	483,403.55	221,262.73	31.2	92.8	124.0	524,243.32	Utah
Vermont	301.6	1,291,829.51	529,907.38	365,064.10	30.5	4.7	35.2	99,416.48	25,668.93	10,786.65	2.3		2.3	52,345.06	Vermont
Virginia	1,624.7	6,193,760.18	2,810,606.22	1,457,161.72	260.9	46.2	307.1	497,760.77	246,236.51	6,200.00	18.4	.6	19.0	578,394.71	Virginia
Washington	1,001.4	5,833,992.87	2,590,959.02	1,168,780.48	175.7	26.8	202.5	774,880.71	326,442.43	61,308.65	12.7	17.1	29.8	893,899.52	Washington
West Virginia	772.6	5,156,102.15	2,037,767.87	693,627.31	129.9	12.5	142.4	626,633.87	274,083.33	135,830.00	9.1	12.9	22.0	334,335.98	West Virginia
Wisconsin	2,414.8	8,618,261.82	3,763,295.04	1,806,000.00	228.9	74.5	303.4	1,690,746.84	552,719.04	182,211.21	48.9	17.6	66.5	62,080.01	Wisconsin
Wyoming	1,629.0	4,684,178.27	2,794,003.06	899,674.51	351.0	242.4	593.4	311,033.86	155,516.00	115,600.00	10.7	43.9	54.6	162,707.71	Wyoming
Hawaii	47.6	1,429,014.40	580,743.37	394,528.83	41.1		41.1	82,324.99	22,869.00		1.5		1.5	1,579,975.17	Hawaii
Totals	88,713.1	387,396,991.92	172,587,244.08	67,308,514.17	12,305.8	4,174.0	16,479.8	60,165,458.60	26,885,946.86	8,470,225.64	1,945.6	1,033.3	2,978.9	39,638,888.04	Totals

(*) The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE: 1931